30799

National Library of Canada | Bibliothèque nationale du Canada

CANADIAN THESES ON MICROFICHE | THÈSES CANADIENNES SUR MICROFICHE

NAME OF AUTHOR/*NOM DE L'AUTEUR* DENNIS E. PUFAHL

TITLE OF THESIS/*TITRE DE LA THÈSE* THE BEHAVIOR OF THAWING SLOPES IN PERMAFROST

UNIVERSITY/*UNIVERSITÉ* ALBERTA

DEGREE FOR WHICH THESIS WAS PRESENTED/*GRADE POUR LEQUEL CETTE THÈSE FUT PRÉSENTÉE* Ph. D

YEAR THIS DEGREE CONFERRED/*ANNÉE D'OBTENTION DE CE DEGRÉ* 1976

NAME OF SUPERVISOR/*NOM DU DIRECTEUR DE THÈSE* N. R. MORGENSTERN

Permission is hereby granted to the NATIONAL LIBRARY OF CANADA to microfilm this thesis and to lend or sell copies of the film.

The author reserves other publication rights, and neither the thesis nor extensive extracts from it may be printed or otherwise reproduced without the author's written permission.

L'autorisation est, par la présente, accordée à la BIBLIOTHÈQUE NATIONALE DU CANADA de microfilmer cette thèse et de prêter ou de vendre des exemplaires du film.

L'auteur se réserve les autres droits de publication; ni la thèse ni de longs extraits de celle-ci ne doivent être imprimés ou autrement reproduits sans l'autorisation écrite de l'auteur.

DATED/*DATÉ* Sept 13/76 SIGNED/*SIGNÉ* D Pufahl

PERMANENT ADDRESS/*RÉSIDENCE FIXÉ* Dept. of Civil Eng
University of Sask.
Saskatoon Sask.

NL-91 (11-78)

INFORMATION TO USERS

THIS DISSERTATION HAS BEEN
MICROFILMED EXACTLY AS RECEIVED

This copy was produced from a microfiche copy of the original document. The quality of the copy is heavily dependent upon the quality of the original thesis submitted for microfilming. Every effort has been made to ensure the highest quality of reproduction possible.

PLEASE NOTE: Some pages may have indistinct print. Filmed as received.

Canadian Theses Division
Cataloguing Branch
National Library of Canada
Ottawa, Canada K1A ON4

AVIS AUX USAGERS

LA THESE A ETE MICROFILMEE
TELLE QUE NOUS L'AVONS RECUE

Cette copie a été faite à partir d'une microfiche du document original. La qualité de la copie dépend grandement de la qualité de la thèse soumise pour le microfilmage. Nous avons tout fait pour assurer une qualité supérieure de reproduction.

NOTA BENE: La qualité d'impression de certaines pages peut laisser à désirer. Microfilmee telle que nous l'avons reçue.

Division des thèses canadiennes
Direction du catalogage
Bibliothèque nationale du Canada
Ottawa, Canada K1A ON4

THE UNIVERSITY OF ALBERTA

THE BEHAVIOR OF THAWING SLOPES IN PERMAFROST

by

© DENNIS E. PUFAHL

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

AND RESEARCH IN PARTIAL FULFILMENT OF THE

REQUIREMENTS FOR THE DEGREE OF

DOCTOR OF PHILOSOPHY

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

FALL, 1976

THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled "THE BEHAVIOR OF THAWING SLOPES IN PERMAFROST" submitted by Dennis E. Pufahl in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Civil Engineering.

.....
N.R. Morgenstern, Supervisor

.....
S. Thomson

.....
D. Cruden

.....
R. Gilpin

Date: September 1, 1976

.....
J.R. Mackay, External Examiner

ABSTRACT

Slope instability is one of the major considerations in the choice of route location across terrain underlain by permafrost. A field reconnaissance has established the state-of-the-art of cut slope design, construction and performance in the Western Canadian Arctic and Alaska. It is apparent that limited instability of backslope cuts in fine grained frozen soils may be encountered in almost all of these regions. Emphasis has been placed on establishing the geological history, the land form, and the associated soil and ground ice characteristics.

Flow dominated failures on natural slopes and cut slopes are common in regions of permafrost. A means of stabilizing these failures is proposed. The method utilizes insulation to control the rate of thaw and the generation of excess pore water pressures. A sand/gravel surcharge load helps to stabilize the slope by increasing the normal effective stress disproportionately to the shearing stress. Design charts are provided and recommendations for installation are included.

Uncontrolled thawing of backslope cuts in fine grained ice rich soil generally results in the formation of bimodal flows common to periglacial regions. A prerequisite for the development of rational methods of control of these features is the establishment of the energetics of exposed melting permafrost. The theory is developed and verified by field experiments and thus provides the basis for controlling the rate of thaw of exposed permafrost.

Recommendations for the design and construction of cut slopes in frozen soils are outlined. Some control methods are suggested that incorporate sand/gravel and other types of higher quality insulation in a variety of different configurations.

ACKNOWLEDGEMENTS

The activities and research presented in this thesis were carried out under the direction of Dr. N.R. Morgenstern, University of Alberta. The author is grateful and indebted to him for his suggestions regarding the topic, for his help in interpreting and presenting the data, and especially for his seemingly limitless enthusiasm for the research project. The help of Dr. Stan Thomson was also warmly received.

Sincere appreciation is extended to the University of Alberta support staff which include Messrs. O. "Woody" Woods, Roy Gitzel, Al Muir, and Don Nordheimer.

This research project was sponsored by the Department of Energy, Mines and Resources, the Defense Research Board, and the National Research Council of Canada. The Department of Public Works, Government of Can., Northwest Territories and the Yukon Territory, the Alaska Department of Highways in Fairbanks, and the State Materials Laboratory in College Alaska all receive recognition for their suggestions, participation and interest in the project.

Personal financial assistance from the Transportation Development Agency, the Roads and Transportation Association of Can., and the North American Life Canadian Council of Professional Engineers was gratefully received.

In addition the author wishes to acknowledge the support and encouragement offered by wife, Idoline Ann, during the first year and a half in Edmonton. Sincere apologies are extended to her for any economic, social or emotional deprivation that may have resulted from this undertaking.

Finally, the unflaging interest and help from Mr. Bill Roggensack during those endless miles of dust and rain and gravel will always be remembered. His editorial assistance was invaluable and the personal friendship and support provided by him and his wife, Muriel, made the completion of this thesis possible.

TABLE OF CONTENTS

		Page
Release Form		i
Title Page		ii
Approval Sheet		iii
Abstract		iv
Acknowledgements		v
Table of Contents		vii
List of Tables		xv
List of Figures		xvi
CHAPTER I	INTRODUCTION	1
	1.1 General	1
	1.2 Scope of Thesis	3
CHAPTER II	BEHAVIOR OF EXISTING CUTS IN PERMAFROST	5
	2.1 Objectives	5
	2.2 Route	6
	2.3 Alaska Highway [Mile 222 (from Dawson Creek, B.C.) 57°58'N latitude; 122°46' W longitude]	7
	2.3.1 Summary	8
	2.4 Dempster Highway	10
	2.4.1 Location and Background	10
	2.4.2 Mile 0 - Mile 80	12
	2.4.3 Mile 80 - Mile 154	14
	2.4.4 Mile 154 - Mile 178	16
	2.4.5 Fort McPherson to Arctic Red River Mile 330 to Mile 365	18
	2.4.6 Mile 178 - Mile 330	19

		Page
	2.5 Mackenzie Highway (Inuvik to Arctic Red River)	19
	2.6 Alaska Reconnaissance	20
	2.6.1 General	20
	2.6.2 TAPS Haul Road	20
	2.6.3 Copper River Basin	23
	2.7 Conclusions	24
CHAPTER III	STABILIZATION OF PLANAR LANDSLIDES IN PERMAFROST	33
	3.1 Introduction	33
	3.2 Thaw Consolidation Pore Water Pressures	33
	3.3 Stability Analysis of Infinite Thawing Slopes	37
	3.4 Numerical Procedure for Design	41
	3.5 Design Charts	46
	3.6 Example Problem	47
	3.7 Recommendations for Installation	53
	3.8 Long term Performance	55
	3.9 Choice of Insulation	55
CHAPTER IV	THE ENERGY BALANCE	70
	4.1 Basic Concepts	70
	4.2 Radiative Components on a Horizontal Surface	71
	4.2.1 General	71
	4.2.2 Direct Beam Clear Sky Solar Radiation	73
	4.2.3 Diffuse Clear Sky Radiation	76

Page

4.2.4 The Effect of Clouds 79

4.2.5 Summary of Short Wave Radiation 83

4.2.6 Short Wave Radiation Reflected by the Surface of the Earth 83

4.2.7 Net Short Wave Radiation 84

4.2.8 Longwave Radiation Emitted by The Ground Surface 86

4.2.9 Longwave Radiation From the Atmosphere to the Ground 88

4.2.10 Net Longwave Radiation 93

4.3 Radiative Components on a Sloping Surface 98

4.3.1 General 98

4.3.2 Direct Beam Radiation 99

4.3.3 Diffuse Radiation 103

4.3.4 Short Wave Component Reflected to the Surface by Surrounding Surfaces 106

4.3.5 Short Wave Component of Radiation Reflected by the Surface 108

4.3.6 Downward Longwave Component of Radiation Falling on the Slope 109

4.3.7 Downward Longwave Component Reflected to the Surface by Surrounding Surfaces 110

4.3.8 Upward Component of Long Wave Radiation Reflected onto the Slope by Surrounding Surfaces 110

4.3.9 Long wave Radiation Emitted by the Surface 111

4.3.10 Summary of Components 111

4.4 Turbulent Transfer Processes 114

4.5 Evaporation and Condensation 114

		Page
	4.6 Sensible Heat Transfer	120
	4.7 Energy Transfer Due to Precipitation	126
	4.8 Soil Heat Transfer	127
	4.9 Summary of Heat Balance	128
CHAPTER V	HEAT FLUX MEASUREMENTS AT THE HEAD SCARP	129
	5.1 Introduction	129
	5.2 Site Location and Description	129
	5.3 Ablation Rates	142
	5.3.1 Measurement Techniques	142
	5.3.2 Magnitude and Heat Requirements	144
	5.4 Radiation Balance	151
	5.4.1 Short Wave Radiation	151
	5.4.2 Net Short Wave Radiation	152
	5.4.3 Net Long Wave Radiation	152
	5.4.4 Net All wave Radiation	156
	5.5 Radiation Field Instrumentation	156
	5.6 Data Acquisition System	157
	5.7 Measured Total Short Wave Radiation	162
	5.8 Measured Net Radiation	166
	5.9 Latent Heat Transfer	173
	5.10 Evaporation Quantities	185
	5.11 Sensible Heat Transfer	187
	5.12 Ablation and the Heat Balance	189

		Page
CHAPTER VI	CONTROL OF THAWING IN EXPOSED PERMAFROST	205
	6.1 Head Scarp Geometry of Bimodal Flows	205
	6.2 Basis for Control	206
	6.3 Suggested Methods of Control	209
	6.4 Recommended Practice for Cut Slope Design	214
	6.4.1 Self Healing Cut Slopes	214
	6.4.2 Non Self-Healing Cut Slopes	217
CHAPTER VII	CONCLUDING REMARKS	222
LIST OF REFERENCES		227
APPENDIX A	DETAILS OF FIELD RECONNAISSANCE	236
	A.1 Alaska Highway (Mile 222 (from Dawson Creek, B.C.) 57°58N Latitude; 122°46'W Longitude)	236
	A.1.1 Physiography	236
	A.1.2 Climate	236
	A.1.3 Permafrost	236
	A.1.4 Bedrock Geology	237
	A.1.5 Surficial Geology	237
	A.1.6 Highway Details (Mile 222)	238
	A.2 Dempster Highway	238
	A.2.1 Dempster Highway (Mile 0 to Mile 80)	239
	A.2.2 Climate	239
	A.2.3 Permafrost	240
	A.2.4 Bedrock Geology	240
	A.2.5 Surficial Geology	240

		Page
A.2.6	Highway Details	240
A.3	Dempster Highway (Mile 80 to Mile 154)	241
A.3.1	Physiography	241
A.3.2	Climate	242
A.3.3	Permafrost	242
A.3.4	Bedrock Geology	242
A.3.5	Surficial Geology	242
A.3.6	Highway Details	243
A.3.6.1	Highway Details (Mile 80 - Mile 96)	243
A.3.6.2	Highway Details (Mile 96 to Mile 123)	243
A.3.6.3	Highway Details (Mile 114)	244
A.3.6.4	Highway Details (Mile 120.7)	244
A.3.6.5	Highway Details (Mile 123 to Mile 154)	244
A.3.7	Summary	245
A.4	Dempster Highway (Mile 154 to Mile 178)	245
A.4.1	Physiography	245
A.4.2	Climate	246
A.4.3	Permafrost	246
A.4.4	Bedrock Geology	246
A.4.5	Surficial Geology	246
A.5	Highway Details	247
A.5.1	Highway Details (Mile 168)	247
A.5.2	Highway Details (Mile 175)	247
A.5.3	Summary	248

		Page
A.6	Dempster Highway (Fort McPherson to Arctic Red River, Mile 330 to Mile 365)	249
	A.6.1 Physiography	249
	A.6.2 Climate	249
	A.6.3 Permafrost	249
	A.6.4 Bedrock Geology	249
	A.6.5 Surficial Geology	249
	A.6.6 Highway Details	250
A.7	Dempster Highway (Mile 178 to Mile 330)	251
	A.7.1 Physiography	251
	A.7.2 Climate	251
	A.7.3 Permafrost	251
	A.7.4 Bedrock Geology	251
	A.7.5 Surficial Geology	252
	A.7.6 Observations	252
	A.7.7 Summary	252
A.8	Mackenzie Highway (Inuvik to Arctic Red River)	253
A.9	Alaska Reconnaissance	253
	A.9.1 General	253
	A.9.2 Physiography	254
	A.9.3 Climate	255
	A.9.4 Permafrost and Vegetation	255
	A.9.5 Bedrock Geology	255
	A.9.6 Surficial Geology	256
	A.9.7 Highway Details	256
	A.9.7.1 Highway Details (Mile 14)	256

Page

A.9.7.2 Highway Details (Mile 19.6) 257

A.9.7.3 Highway Details (Mile 20.3 to Mile 20.5) 257

A.9.7.4 Highway Details (Mile 22.9) 257

A.9.7.5 Highway Details (Mile 23.4) 258

A.9.7.6 Highway Details (Mile 33.0 to Mile 33.4) 259

A.9.7.7 Highway Details (Mile 42.8) 259

A.9.7.8 Highway Details (Mile 53.5) 259

A.9.7.9 Highway Details (Mile 54.4) 259

A.9.8 Summary 260

A.10 Copper River Basin 260

A.10.1 Physiography 260

A.10.2 Climate 261

A.10.3 Permafrost 261

A.10.4 Bedrock Geology 261

A.10.5 Surficial Geology 261

A.10.6 Highway Details 261

A.11 Conclusions 262

APPENDIX B COMPUTER PROGRAM AND EXAMPLE PROBLEM OF SLOPE STABILIZATION TECHNIQUE 272

APPENDIX C

C.1 Computer program to calculate the length of a shadow cast by an obstruction on sloping ground 289

		Page
	C.2 Computer program to compute the direct beam and total solar daily radiation on a sloping surface from values monitored on a horizontal surface	231
	C.3 Procedure for calculating the amount of reflected diffuse radiation from one surface to another	292
APPENDIX D		308
	D.1 Computer program for computing vapor pressures, vapor pressure gradients and temperature gradients from wet and dry bulb temperatures	308
	Radiation data in Table D-1	

LIST OF TABLES

Table		Page
2.1	Summary of Soil Data	29
2.2	Summary of Cuts	30
3.1	Physical and Thermal Properties of Various Insulating Methods	42
3.2	Soil, Insulation and Climate Information for Example Problem	48
3.3	Thaw Depth and Insulation and Surcharge Requirements for Example Problem	51
3.4	Comparison of Factors of Safety by Different Methods of Computation	52
4.1	Albedo of Various Surfaces	85
4.2	Infrared Emissivities	87
4.3	Air Temperature, Relative Humidity, and Vapour Pressure	118
4.4	Snow Melt Due to Sensible & Convective Heat Transfer and Bowen's Ratio (After U.S. Army Corps of Eng., 1956)	124
5.1	Summary of Soil Data at Fort Simpson Landslide	139
5.2	Short Wave Radiation Quantities for Slopes at Different Orientations and Inclinations for Fort Simpson Region	195
5.3	Net Longwave Radiation for Different Slopes Computed from Equation 4.53	197(a)
5.4	Net Radiation for Slopes at Different Orientations and Inclinations for Fort Simpson Region	198
5.5	Daily and Accumulated Total Radiation on a Horizontal Surface at the Fort Simpson Test Site	200
5.6	Net Radiation at the Fort Simpson Test Site	201
5.7	Comparison of Predicted Values and Measured Values of Net Radiation	203
5.8	Lysimetric Data	204
D.1	Radiation Data (Fort Simpson Test Site)	

LIST OF FIGURES

Figure		Page
2.1	Transportation Facilities - 1972 Northwestern Canada	8
2.2	Permafrost in Canada (After Brown, 1967)	9
2.3	Location of the Dempster Highway, Dawson City, Yukon Territory to Arctic Red River, N.W.T.	11
2.4	Physiographic Regions of the Yukon Territory	13
2.5	Location of Mile 90 to Mile 180 of the Dempster Highway	15
2.6	TAPS Haul Road; Livengood to the Yukon River Alaska	21
3.1	Thaw Depth vs Time and Thaw Depth vs $\sqrt{\text{Time}}$ for an Insulated Thawing Slope	36
3.2	Pore Pressure at the Thaw Front of a Thawing Infinite Slope	38
3.3	Balance of Forces on a Thawing Infinite Slope	38
3.4	Pore Pressure Factor (R') for Different Thaw Consolidation Ratios (R) After Morgenstern & Nixon (1971)	40
3.5	Pore Pressure Factor (R") for Different Thaw Consolidation Ratios (R) After Morgenstern & Nixon (1971)	40
3.6	Design Chart for Stabilization of Planar Landslides	58
3.7	Design Charts for Stabilization of Planar Landslides	59
3.8	Design Charts for Stabilization of Planar Landslides	60
3.9	Design Charts for Stabilization of Planar Landslides	61
3.10	Design Charts for Stabilization of Planar Landslides	62
3.11	Design Charts for Stabilization of Planar Landslides	63
3.12	Design Charts for Stabilization of Planar Landslides	64
3.13	Design Charts for Stabilization of Planar Landslides	65
3.14	Design Charts for Stabilization of Planar Landslides	66
3.15	Design Charts for Stabilization of Planar Landslides	67
3.16	Design Charts for Stabilization of Planar Landslides	68

Figure		Page
3.17	Design Charts for Stabilization of Planar Landslides	69
4.1	Monochromatic Emissive Power for Several Temperatures vs Wavelength (After Wiebelt (1966))	72
4.2	Atmospheric Transmission Chart (After Kimball 1928)	78
4.3	Extraterrestrial Daily Radiation Received on a Horizontal Surface (After Lui and Jordan, 1960)	80
4.4	The Ratio of the Monthly Ave. Daily Diffuse Radiation to the Monthly Ave. Daily Total Radiation as a Function the Cloudiness Index K_T (After Lui & Jordan, 1960)	80
4.5	Ratio of Direct to Diffuse Radiation Received at the Ground Surface (%) After London (1957)	81
4.6	Saturated Vapour Pressure vs Temperature	117
4.7	Convection Melt (computed) (After U.S. Army Corps of Eng., 1956)	123
4.8	Condensation Melt (computed) (After U.S. Army Corps of Eng., 1956)	123
5.1	Aerial View of Fort Simpson Landslide	130
5.2	Bank Profile of Mackenzie River Immediately East of Fort Simpson Landslide	131
5.3	Plan View of Fort Simpson Landslide	133
5.3(a)	Description of Sediments of Fort Simpson Landslide	133(a)
5.4	Aerial View of East Flank of Fort Simpson Landslide	134
5.5	Plan View and Instrumentation of Headscarp of Bimodal Flow at Fort Simpson Landslide	136
5.6	View of Headscarp (July 1974)	137
5.7	Mudflow at the Base of the Headscarp	137
5.8	Soil Profile Zone 1, 2 & 3 (Oct. 1973)	141
5.9	Soil in Zone 3	141
5.10	Frozen Core Removed From Zone 4	141
5.11	Schematic Arrangement of Ablation Measurement Technique	143

Figure		Page
5.12	Ablation Measurement Technique	145
5.13	Accumulated Ablation at Sta. 1	145
5.14	Accumulated Ablation at Sta. 2	146
5.15	Accumulated Ablation at Sta. 3	146
5.16	Accumulated Ablation at Sta. 4	147
5.17	Accumulated Ablation at Sta. 5	147
5.18	Accumulated Ablation at Sta. 6	148
5.19	Water Content vs. Relative Frequency Curve	148
5.20	Cos Λ vs. Time for a Sloping Surface	153
5.21	Cos Λ vs. Time for a Horizontal Surface	153
5.22	Net Radiometer (Fritchen Model)	158
5.23	Net Radiometers Around Headscarp	158
5.24	Typical Variation of Radiation Intensity with Time	160
5.25	Schematic Sketch of Radiation Instrumentation	165
5.26	Data Acquisition System	165
5.27	Total Shortwave Radiation on a Horizontal Surface for the Period of Observation	165
5.28	Net Radiation for the Period of Observation (Sta. 1)	167
5.29	Net Radiation for the Period of Observation (Sta. 2)	168
5.30	Net Radiation for the Period of Observation (Sta. 3)	169
5.31	Net Radiation for the Period of Observation (Sta. 4)	170
5.32	Net Radiation on a Horizontal Surface for the Period of Observation (Sta. 5)	171
5.33	Accumulated Net Radiation for the Period of Observation	172

Figure		Page
5.34	Permafrost Lysimeter	175
5.35	Permafrost Lysimeter	176
5.36	Tray from Permafrost Lysimeter (weighed with standard load cell)	176
5.37	Portable Crane and Permafrost Lysimeter	177
5.38	Product of Average Wind Speed and Relative Humidity vs. Latent Heat of Condensation	180
5.39	Wet and Dry Bulb Temperature Sensors	182
5.40	Typical Vapour Pressure Gradients Above the Headscarp	184
5.41	Daily Evaporation Rates at Test Site	186
5.42	Typical Temperature Gradients above the Headscarp	188
5.43	Heat Balance at the Headscarp of the Fort Simpson Landslide	191
6.1	Gravel Thickness, Ground Surface Thawing Index and Headscarp Retreat for Ice	212
6.2	Gravel and Artificial Insulation Configuration for Steep Slopes	218
6.3	Gravel and Artificial Insulation Configuration for Moderately Steep Slopes	218
6.4	Gravel and Rock Filled Gabions with Artificial Insulation on Moderately Steep Slope	220
6.5	Gravel and Artificial Insulation on Low Angle Slope	220
A.1	Mile 222 Alaska Highway (Backslope instability during construction, July 1973).	263
A.2	Mile 50 Dempster Highway (Headwaters of the East Blackstone River. Road built at very modest geometric design standards)	263
A.3	Mile 124 Dempster Highway (Typical view of the Ogilvie physiographic unit)	263
A.4	Mile 80 Dempster Highway (Improved geometric design standards)	264

Figure		Page
A.5	Mile 109 Dempster Highway (Biangular profile of backslope due to permafrost degradation)	264
A.6	Mile 114 Dempster Highway (Large thermal degradation scar - 150 ft. wide and 15 to 20 ft. deep)	264
A.7	Mile 120.7 Dempster Highway (Ten ft. cut with 2:1 backslopes made into ice rich organic silt)	265
A.8	Mile 153 Dempster Highway (Melting of large ice wedges caused slope degradation)	265
A.9	Mile 153 Dempster Highway (View from top dressed backslope)	266
A.10	Mile 175 Dempster Highway (Backslope and shoulder failure due to melting of a large ice lense)	266
A.11	Mile 175 Dempster Highway (Severe cracking of shoulder due to differential thermal subsidence caused by construction	266
A.12	Mile 343 Dempster Highway - Large cut between Arctic Red River and Fort McPherson (looking west)	267
A.13	Mile 343 Dempster Highway - Stabilized backslope. (Hand placed vegetation to control rate of thaw)	267
A.14	Bimodal flow in the Richardson Mountains	267
A.15	Approach cut to Mackenzie River crossing at Arctic Red River	268
A.16	Typical vegetation patterns at southern end of TAPS Haul Road	268
A.17	Mile 14 TAPS Haul Road (July 1973)	268
A.18	Mile 20 TAPS Haul Road - Typical Bore Hole Log (After Smith & Berg 1972)	269
A.19	Mile 1916 (31.6 km) TAPS Haul Road (July 1973)	269
A.20	Mile 20.3 (32.6 km) TAPS Haul Road (July 1973)	269
A.21	Mile 22.9 TAPS Haul Road (April 1970) (After Smith & Berg 1972)	270
A.22	Mile 22.9 TAPS Haul Road (July 1973)	270
A.23	Mile 23.4 TAPS Haul Road. Approach to Hess Creek (April 1970)	270

Figure		Page
A.24	Mile 23.4 TAPS Haul Road. Approach to Hess Creek (July 1973)	271
A.25	Mile 33 TAPS Haul Road (July 1973)	271
A.26	Mile 54.4 TAPS Haul Road (July 1973)	271
C.1	Shadow Cast by Obstruction on Sloping Ground	290
C.2	Relation of a Small Surface dA_2 to a Diffusely Reflecting Large Surface A_1 (After Threlkeld, 1963)	292

CHAPTER I

INTRODUCTION

1.1 General

The increased demand for energy and raw materials within the last decade has resulted in the exploitation of many frontier reserves of minerals, oil, and natural gas. The Canadian and Alaskan Arctic are presently the center for much of this activity. The development of natural resources in these regions requires the construction of various transportation facilities over terrain underlain by discontinuous and continuous permafrost. One of the major problems associated with route location for these facilities is that of slope instability.

Research activities by Capps (1919), Eakin (1919), Taber (1943), and Sigafoos and Hopkins (1952) provided a substantial quantity of qualitative information regarding slope instability in periglacial regions. The proposed pipeline construction in the Mackenzie valley added new impetus to this aspect of Arctic research. Hardy and Morrison (1972), Hughes (1972), Issacs and Code (1972) and MacKay and Mathews (1973) provided greater insight into the mechanisms of mass movements in the arctic regions.

McRoberts (1973) classified the different types of mass movements encountered along the Mackenzie River valley and its tributaries. He successfully attempted to explain the mechanisms of failure of thawing permafrost slopes in quantitative geotechnical terms. Thaw consolidation excess pore water pressures due to self weight were included in his analysis of an infinite thawing slope. This equation was

$$F_s = \gamma'/\gamma \left(1 - \frac{1}{1 + \frac{1}{2R^2}}\right) \frac{\tan \phi'}{\tan \theta} \quad - - - [1.1]$$

(After McRoberts and Morgenstern, 1974)

where F_s = factor of safety of the slope

γ' = effective unit weight of the soil

γ = total unit weight of the soil

ϕ' = angle of internal friction of the soil

θ = angle of the slope

R = thaw consolidation ratio

This relationship adequately explained soil movements on low angle slopes that would have otherwise been stable if only steady state seepage pore water pressures had been present. It was apparent that any stabilization techniques would require the alteration of the thaw-consolidation ratio (R) which includes both thermal and physical properties of the soil as well as the imposed step temperature at the surface. This thesis proposes a method for stabilizing planar landslides on thawing slopes.

The geotechnical implications of uncontrolled retreat of bimodal flows was recognized and attention was drawn to the hazards involved with artificial cuts in ice rich soil (ibid). A simple model used to relate the rate of ablation to heat flux at the surface is of the form:

$$V = F/L \quad - - - [1.2]$$

(Carslaw and Jaeger, 1947)

where V = the steady state velocity of the ablation surface

F = the heat flux available at the surface

L = the volumetric latent heat of fusion of the soil/ice mixture

This relationship indicated that a heat flux of approximately 200 to 600 ly/day was required to sustain typical rates of ablation that had been observed in the field. A comparison of these values with synoptic estimates of net radiation indicated a substantial disparity. There appeared to be an unaccountable source of heat involved in the ablation process. Kerfoot (1969) and McRoberts (1973) also noted that bimodal flows did not favor any particular aspect and that maximum retreat did not always occur on slopes with the most southerly aspect.

The questions concerning the high rate of ablation, the indifference to aspect, and the unique shape of the headscarp of bimodal flows remained unanswered. A complete analysis of the heat balance at the surface of exposed melting permafrost was considered necessary to properly explain the observed phenomena and to provide a rational basis for the design of remedial measures.

1.2 Scope of Thesis

Transportation facilities constructed across permafrost terrain necessitate the cutting of slopes in frozen soils that are not stable in the thawed state. A field reconnaissance was undertaken in order to appraise the behavior of existing road cuts. Chapter II outlines the nature of this reconnaissance, summarizes the findings, and contains the conclusions of the survey. The details of this study along with a variety of illustrated examples are contained in Appendix A.

One of the common forms of natural instability involves a surface movement or detachment failure of surficial material on reasonably planar surfaces. These flow dominated movements are evident not

only on natural slopes but on low angle back slope cuts as well. The necessity for some means of positive stabilization of these failures or incipient failures is apparent. Chapter III identifies the important variables involved in this problem and proposes a procedure to remedy this type of instability.

If rational design procedures are to be utilized to prevent excessive degradation of cut slopes or to control naturally occurring bimodal flows it is essential to establish the energetics of exposed melting permafrost. Chapter IV outlines the heat balance theory for a sloping surface of exposed permafrost and postulates the presence of substantial sources of heat other than net radiation. The theory was verified by field experiments which were performed at a large complex landslide on the Mackenzie River near Fort Simpson, N.W.T. The details of the field program and the interpretation of the results appear in Chapter V of this text. The results substantiate the theory and provide the basis for control of thawing of exposed permafrost.

Design and construction procedures for cut slopes are reviewed and a variety of methods for backslope preservation are presented in Chapter VI.

The final chapter contains the concluding remarks and suggests the form of ongoing research related to this aspect of surface transportation facilities in the Arctic.

CHAPTER II

BEHAVIOR OF EXISTING CUTS IN PERMAFROST

2.1 Objectives

Transportation facilities constructed across permafrost terrain often require cuts of varying size to maintain acceptable geometric standards. Exposure of these open cuts to thawing temperatures invariably cause degradation of the permafrost and subsequent deterioration of the backslopes. The magnitude and frequency of this deterioration was previously without documentation with the exception of the work of Smith and Berg (1972) on the Trans-Alaskan Pipeline System Haul Road.

General interrelationships regarding geological history, landform, type of material, and ground ice patterns had been emerging (Mackay and Black, 1973; Hughes, 1974 and others). Furthermore, there had been a mounting log of evidence (McRoberts and Morgenstern, 1973) to indicate that the frequency and scale of natural slope instability increased rapidly in areas containing fine grained ice rich soils. Although slope instability occasionally occurred in the till materials it was not generally characterized by continuing permafrost degradation. It was also evident that the amount and type of ground ice present in the frozen sediments played a major role in the extent and nature of the observed instability.

With these concepts in mind, a field reconnaissance program was undertaken to assess the current state-of-the-art regarding the behavior and development of design concepts pertaining to cut slopes in frozen soils.

The objectives were to establish (whenever possible):

i) the geological, climatic, permafrost and geographical setting;

ii) the soil and ice stratigraphy;

iii) the original cross-section geometry; and,

iv) the existing cross-section geometry.

This information should provide a basis for at least some tentative conclusions regarding the behavior and predicted behavior of cut slopes in arctic and sub-arctic regions under various geographic, climatic, and geologic conditions.

Economic considerations coupled with efficiency and convenience indicated that a small self-contained mobile home, towing a utility trailer, would provide the most versatile means of surface transport. The utility trailer contained a variety of equipment and support items deemed essential for the field work.

2.2 Route

During the course of the reconnaissance approximately 5,500 miles were travelled in 25 days and during that time many dozens of cuts on various highway locations in British Columbia, the Yukon Territory, the Northwest Territories and Alaska were examined, measured and photographed.

The details of these observations are contained in Appendix A while summaries and conclusions appear in the following pages. The information is presented under the sub-headings of Alaska Highway, Dempster Highway, Trans-Alaska Pipeline System Haul Road, and the Copper River Basin. It was felt that the locations could most logically be categorized according to specific projects rather than by territorial or physiographic boundaries. The main sub-divisions will,

therefore, appear in the order in which they were encountered (with minor exceptions).

The major part of the route location is shown in Figure (2-1). The field party started in Edmonton, Alberta on July 4, 1973 and travelled north through Grande Prairie to Dawson Creek, B.C. and then to Whitehorse in the Yukon Terriroty via the Alaska Highway. Travel continued north through the Yukon to Dawson City and up the Dempster Highway to its terminus at Mile 178 (1973). The route crossed the Canada-United States border at Poker Creek west of Dawson City, rejoined the Alaska Highway at Tetlin Junction and continued west to Fairbanks, Alaska. Activities in Alaska dealt primarily with the Trans-Alaska Pipeline System Haul Road, which originates near Livengood, extending fifty-five miles north west to the Yukon River. The Copper River Basin, in south eastern Alaska, was examined by travelling the loop from Fairbanks to Anchorage, east to the Copper River near Glenallen and then north to Tok Junction, and returning to Whitehorse from there.

The north end of the Mackenzie and Dempster Highways were inspected by travelling to Inuvik from Whitehorse by commercial airline and then by helicopter to Arctic Red River, Fort McPherson and the Richardson Mountains.

The field party then returned to Whitehorse from Inuvik and arrived back in Edmonton on July 29, 1973.

2.3 Alaska Highway Mile 222 (from Dawson Creek, B.C.) 57°58' N latitude; 122°46' W longitude

2.3.1 Summary

Concerns in this region regarding the instability of back slope cuts caused by permafrost degradation are slight due to the sporadic occurrence of permafrost (Figure (2-2)). North facing

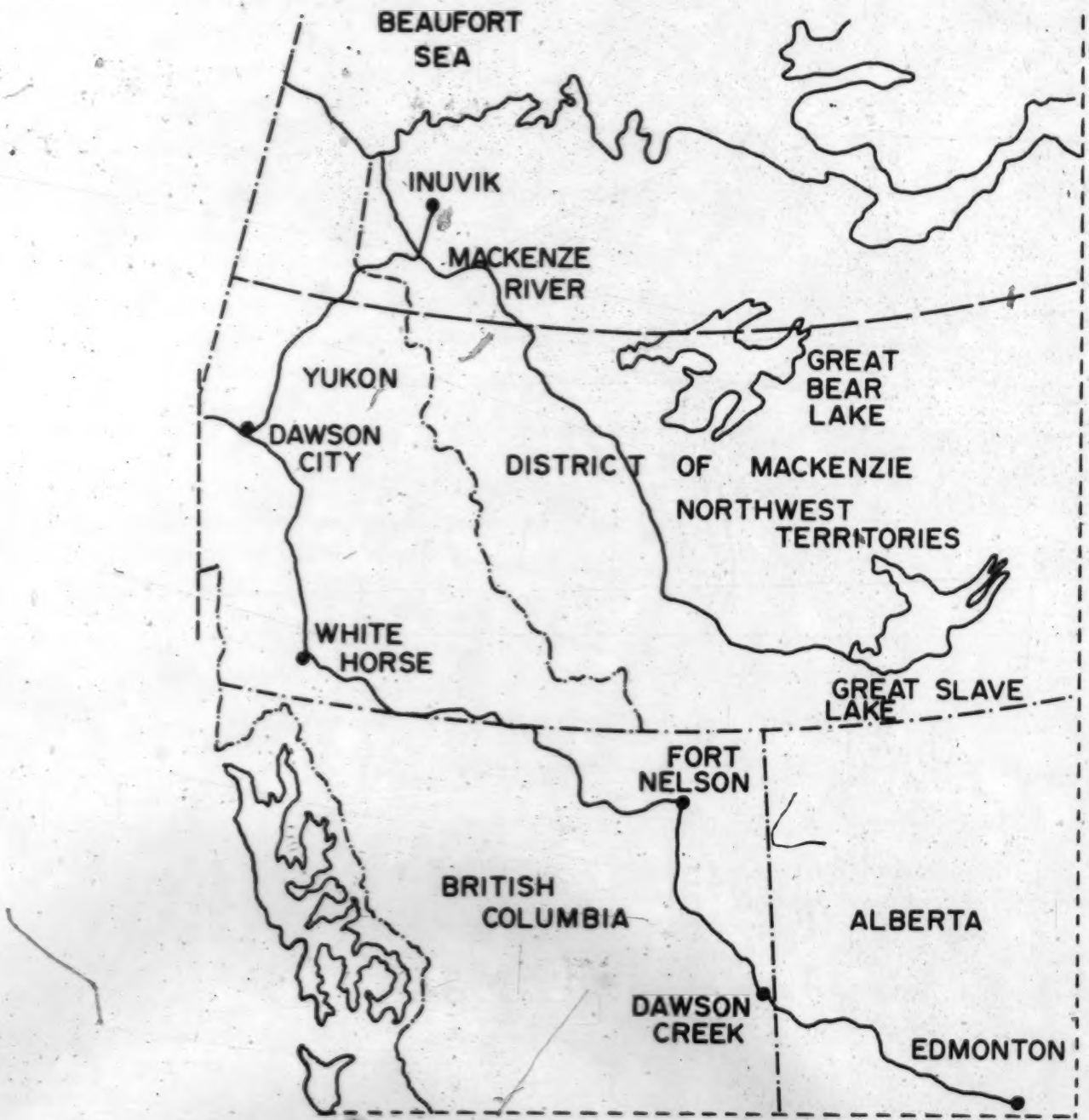

Fig. 2-1 Transportation Facilities - 1972 Northwestern Canada

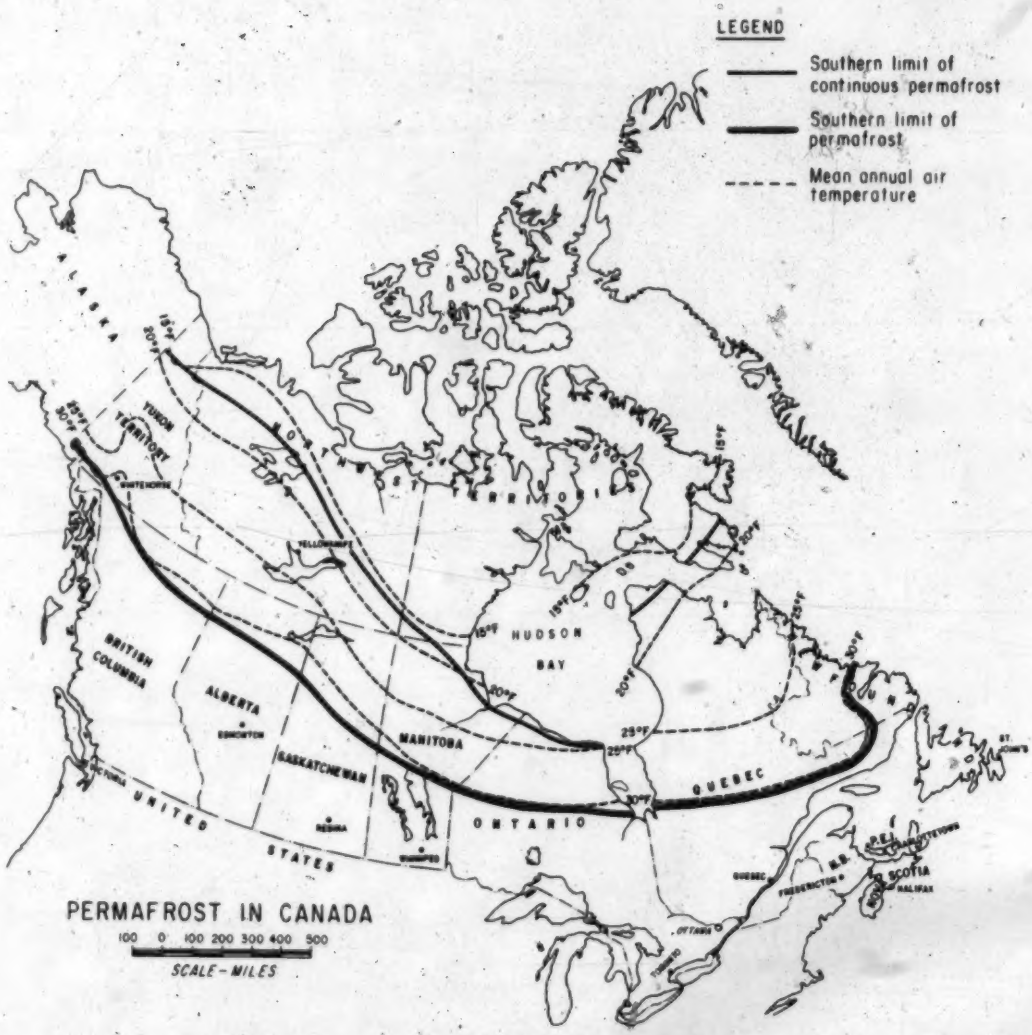

Fig. 2-2 Permafrost in Canada (After Brown, 1967)

slopes covered with heavy stands of black spruce and alder are usually avoided as are low, poorly drained areas. Encounters with permafrost are, therefore, often unexpected and as a result may lead to some particularly troublesome construction situations.

At Mile 222, a highway cut 20 to 25 ft. deep was required to maintain alignment in a reconstruction project. Very wet, fine grained partially frozen material was encountered in the fall of 1972. Cold weather, wet soil, and freezing weather conditions prompted a postponement of construction activities until spring. The material did not melt, drain, and stabilize during the following spring and summer as expected and a continuing problem of back slope instability and a wet subgrade were preventing further construction activity.

The observers felt that if the excavation had been completed and subgrade back fill consisting of select borrow had been installed, then little difficulty other than ditch maintenance would have resulted. It appeared that excavation by dragline and trucks was required to remove the saturated material from the centerline of the roadway. Subsequent back filling with free draining select borrow would provide a stable working base from which to perform ditch maintenance and the remainder of the required excavation.

2.4 Dempster Highway

2.4.1 Location and Background

The Dempster Highway follows an old overland route between the water drainage system of the Yukon and Mackenzie Rivers beginning at Dawson City, Yukon Territory 64°04' N lat., 139°26' W long. via Fort McPherson 67°26' N lat., 135°58' W long. to Arctic Red River 67°27' N lat., 134°44' W long. The location is shown in Figure 2-3.

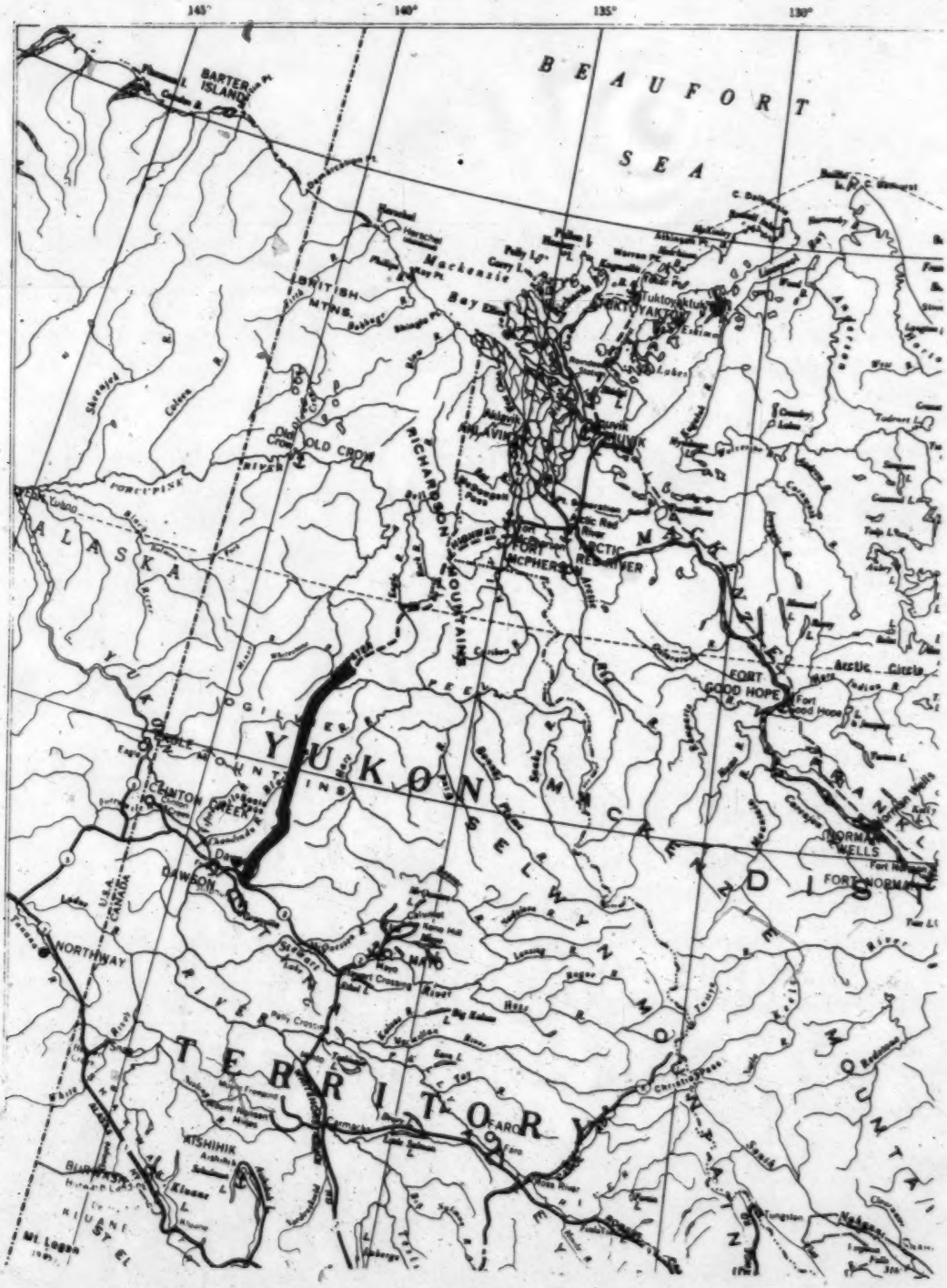

Fig. 2-3 Location of the Dempster Highway; Dawson City, Yukon Territory to Arctic Red River, N.W.T.

Construction of this highway began in 1959 and consisted largely of improving, grading and maintaining an existing tote road. Subsequently, more money was appropriated to the project with the aim of connecting Dawson City on the Yukon River to Fort McPherson on the Peel River and subsequently, the settlement of Arctic River at the confluence of the Mackenzie and Arctic Red Rivers.

The highway lies primarily within the Northern Plateau and mountain area of the Interior System of the Canadian Cordillera. Figure 2-4 indicates the different physiographic units traversed by this transportation facility.

2.4.2 Mile 0 - Mile 80

The first 45 miles of this section of highway is located along the North Klondike River and the remaining portion along the East Blackstone River. Much of this area has been glaciated and there is an abundance of sub-grade material comprised of glacial sediments along the lower portion of the route. Weathered shales and sandstones occasionally outcrop and are used extensively as surfacing materials.

Permafrost is more or less continuous and consequently the borrow pits formed in the active layer are large and shallow. Thermal subsidence of the road is common and has substantially lowered the grade line. As a result, continued maintenance has been required to sustain the road bed.

Road cuts along this eighty mile section of road were not considered significant. They were small (less than ten ft.) and were stable in all cases.

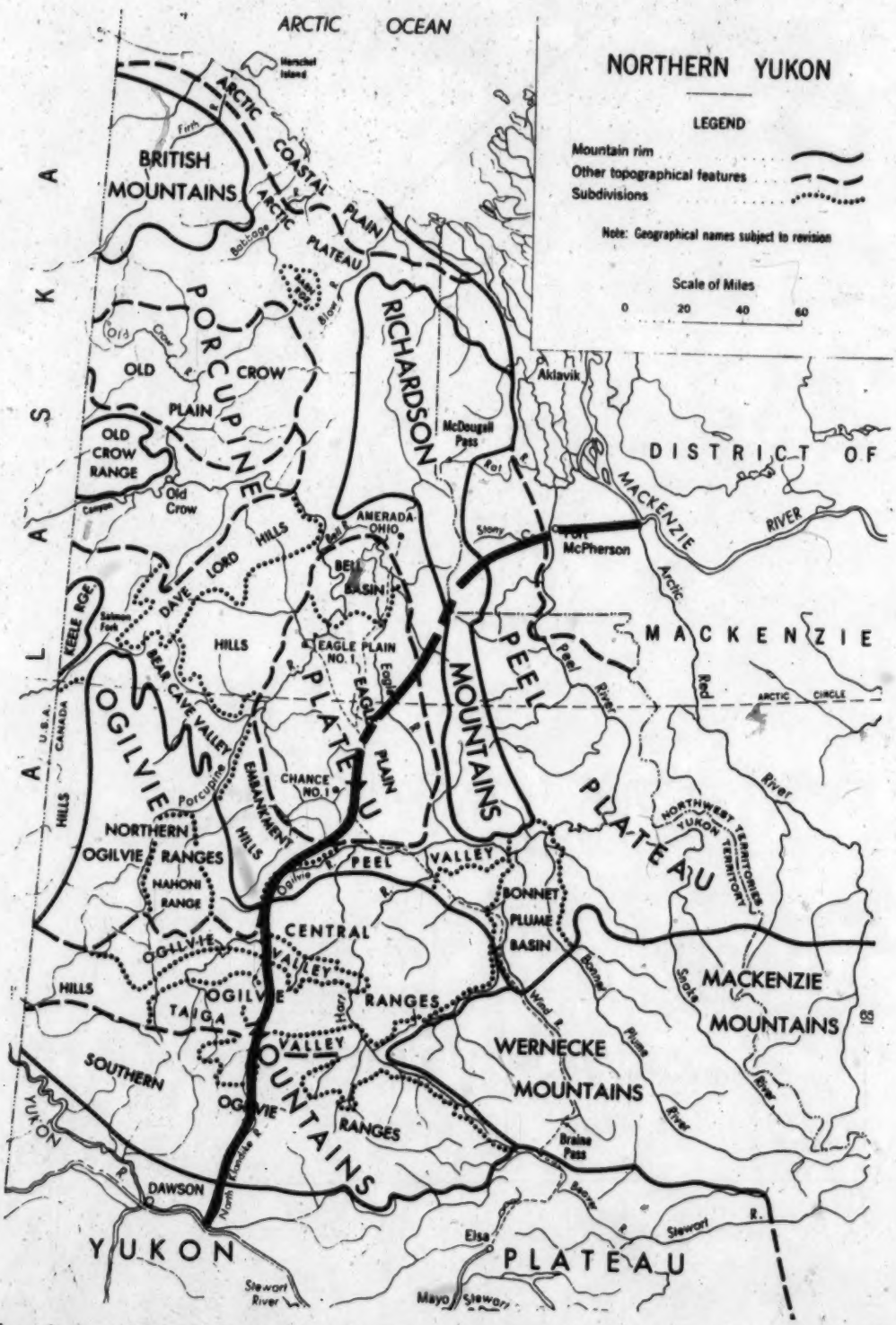

Fig. 2-4 Physiographic Regions of the Yukon Territory (Bostock, 1961)

2.4.3 Mile 80 - Mile 154

The location of this section of road is shown in Figure 2-5. The highway departs from the Blackstone River at Mile 96 and parallels Big Creek, a tributary of the Ogilvie River, to Mile 123 and from there follows the Ogilvie River to Mile 154.

The highway now lies within the unglaciated portion of the Yukon Territory and surficial deposits consist of talus covered slopes, alluvial fan deposits and active and inactive flood plain deposits. Muskeg is abundant in this area and unusually thick deposits of reworked organic matter often cover the coarse gravel of the inactive flood plains. Pediment slopes consisting of frozen colluvium and alluvial silts interspersed with rock detritus and organic matter are frequent along the flanks of the valley.

This lower sixteen mile section of sub-grade shows substantial improvement in alignment and design standards. Back slope cuts are rare, as a major portion of the road is constructed from fill material obtained from the East Blackstone River and Big Creek. The following 20 miles flank the valley walls of Big Creek and the proximity of the river bed has often forced the location onto the sides of the valley. Twenty to thirty foot cuts in colluvial silt and weathered shale are not uncommon. Construction problems and ditch blockage resulted from the melting of ice wedges and frozen soil encountered in the cuts. Small bimodal flows occurred at some locations.

Extensive disturbance was noted on an abandoned tote road located on an alluvial silt or slope wash side hill at Mile 114 directly across the valley from the present highway location. The original activity was estimated to be eight to ten years old. Large craters have formed due to thermal subsidence caused by the subsurface melting of

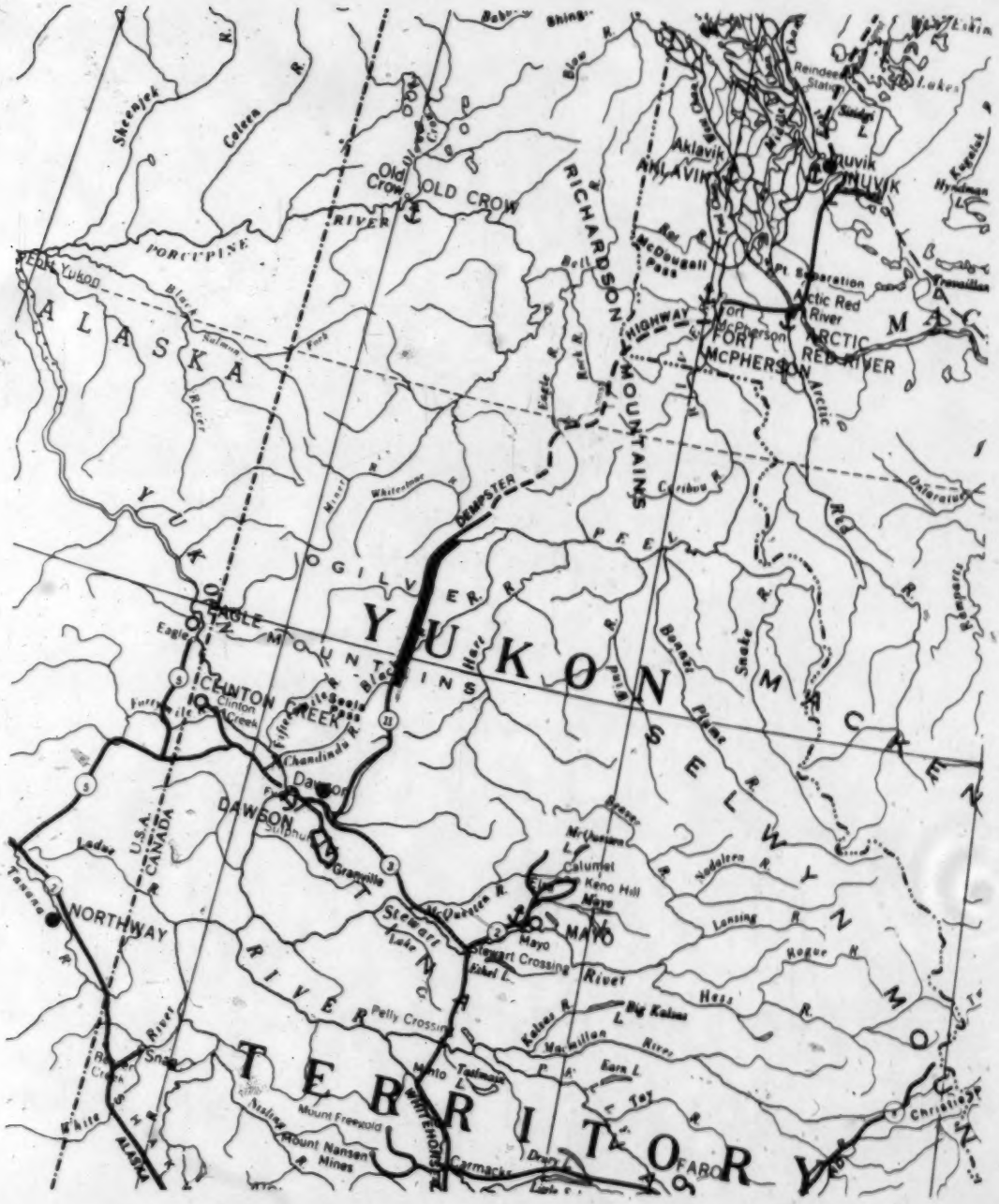

Fig. 2-5 Location of Mile 90 to Mile 180 of the Dempster Highway

large tabular masses of ice.

A significant and troublesome cut occurred at Mile 154 adjacent to the Ogilvie River. A bimodal flow eventually developed with a head scarp thirty feet high. Large ice wedges were exposed in a silt-weathered shale-soil mixture. The melted soil and ice caused a serious impediment to drainage and concern to maintenance personnel. After one complete thaw season, gravel was used to fill the crater formed by the loss of material and thus return the slope to its original dimensions. This remedial measure appears to have adequately prevented further degradation of the slope.

The performance of cuts along this section of highway has generally been good to excellent with little more than ditch clearing required to maintain adequate drainage.

2.4.4 Mile 154 - Mile 178

The Dempster Highway departs from the Ogilvie River and ascends the valley flank to the Eagle Plains. Unglaciated surficial deposits comprise the subgrade construction materials and consists of discontinuous felsenmeer and/or colluvium over bedrock. Extensive deposits of organic matter are common along the lower reaches of this section. The sandstone bedrock is more resistant to weathering and therefore forms the tops of ridges and escarpments. The shales, on the other hand, are softer and weaker and form smoother and more gentle slopes.

Large side hill cuts and steep grades were required to attain the ascent out of the Ogilvie River Valley to the plain above. However, since the roadbed was excavated into ice free sandstone and shale bedrock, no problem was encountered with stability. Excavation of a number of small cuts was performed without major difficulty. By

loosening the frozen soil with a ripper tooth on a large crawler tractor and rapidly excavating with conventional earth moving equipment, it was found possible to construct cuts without encountering excessively wet and muddy working conditions. Occasional encounters with residual ice rich fine grained soils caused construction delays and increased costs. However, the back slopes were not found to exhibit alarming instability.

An excellent opportunity exists to compare the difference and assess the behavior and quality of subgrade constructed in late fall and early summer along this section of road. Subgrade constructed in the spring and early summer tends to undergo substantial differential settlement and cracking as melting proceeds into the active layer during the remainder of the summer. On the other hand, subgrade that was constructed in late summer and fall tends to be more stable and exhibit less differential settlement. This behavior can be attributed to the fact that much of the settlement due to conventional consolidation occurs during the construction period rather than subsequent to the completion of grading.

Mile 178 was the construction terminus on the Yukon portion of the Dempster Highway as of July, 1973. A further 56.5 miles of road is presently being constructed to the Eagle River. No inspection of this road has been made and therefore comment is beyond the scope of this thesis. Preliminary design and location reports indicated that the location would follow summits of the ridges whenever possible and borrow material was to consist almost exclusively of weathered sandstone and shale bedrock except where limited quantities of sand and gravel were available from the Eagle River or other lesser streams convenient to the right of way.

2.4.5 Fort McPherson to Arctic Red River

Mile 330 to Mile 365

This section of highway connecting the settlements of Fort McPherson and Arctic Red River was completed during the summer of 1973. Hummocky or dead ice moraine represent the basic land form which is comprised of deposits of unsorted glacial drift occasionally draped with sorted fine grained sediments that may be water modified or reworked till. Large areas are covered with silt and organic material.

Engineering design problems involved the stability of thawed soils, embankment design and drainage structures. High moisture contents in the upper till precluded its use as a construction material and ripped sandy shales were used almost exclusively as a sub-grade material.

Excavation was generally avoided except for a large cut that was developed in a knoll at Mile 343 in order to supply borrow material for the adjacent sections. However, the material proved to have excessively high water contents and was completely unstable when thawed. Much of this material was wasted in adjacent spoil areas.

During and after completion of the cut, the back slopes exhibited reasonably serious degradation. However, the extent of the retreat was apparently limited to the areal extent of the ice. It appears that the retreat and instability of the back slopes ceased when the excess ice had melted and the water had drained away.

This is the most significant cut on the Dempster Highway and the length and width of the excavation rather than its depth make it an outstanding feature on the landscape. At the time of inspection, the back slopes appeared to have completely stabilized without det-

rimental results.

2.4.6 Mile 178 - Mile 330

A brief helicopter reconnaissance was made of the proposed location of the section of the Dempster Highway between Fort McPherson and south to Mile 178.

Several naturally occurring bimodal flow slides were noted in the Richardson Mountains near the Yukon-Northwest Territories border. Some of these flow slides have run their course or have become dormant while others are currently active. Sediments encountered at those sites consist of significant quantities of ice rich fine grained deposits. None of these slides were in close proximity to the proposed right of way.

2.5 Mackenzie Highway (Inuvik to Arctic Red River)

Two sections of the north end of the Mackenzie Highway received a cursory inspection for the presence of back slope instability. No cuts of significance were encountered in the northernmost Inuvik section. However, the north approach to the Mackenzie River warrants a short discussion as it was the only situation where measures were taken to prevent back slope degradation rather than control it.

A forty foot deep cut had exposed 15 to 20 ft. of ice-rich fine grained sediments overlying interbedded shale and sandstone bedrock. As the excavation proceeded, the ice content of the overburden increased with distance from the river bank. To prevent subsequent degradation, the slope was benched at the bedrock level and the ice-rich fine grained sediments in the upper portion of the slope were

dressed with 10 ft. of crushed shale. The slope appeared to be functioning as anticipated at the time of inspection.

2.6 Alaska Reconnaissance

2.6.1 General

Prior to 1973, the roads in Alaska had been built in the discontinuous permafrost zone except for minor activity on the Arctic Coast. The designers, for the most part, were able to choose their locations judiciously and thus avoid cuts where ice-rich fine grained soils were suspected. Even so, requirements of the oil industry have created a need for road locations that do not allow the designer as much latitude in the choice of route. The Trans-Alaska Pipeline System (TAPS) Haul Road is a typical example of such a problem.

The TAPS Haul Road begins at Livengood some sixty miles northwest of Fairbanks and extends fifty-five miles beyond to the Yukon River (1973). This road has since been completed north to Prudhoe Bay on the Arctic Coast and provides facilities for ground surface transportation of all supplies and personnel recuired in the construction of the Alyeska Pipeline.

The great difficulty associated with the routing of this road was that it, of necessity, crossed the grain of the country. Its origin and destination required the route to climb south facing slopes and descend north facing slopes (Figure 2-6).

2.6.2 TAPS Haul Road

The southernmost section of the TAPS Haul Road, built in 1969-70, crosses broad undulating divides and flat-topped spurs of the Yukon-Tanana Upland and descends to the Yukon River Lowland which exhibits gently rolling topography with maximum relief in the order

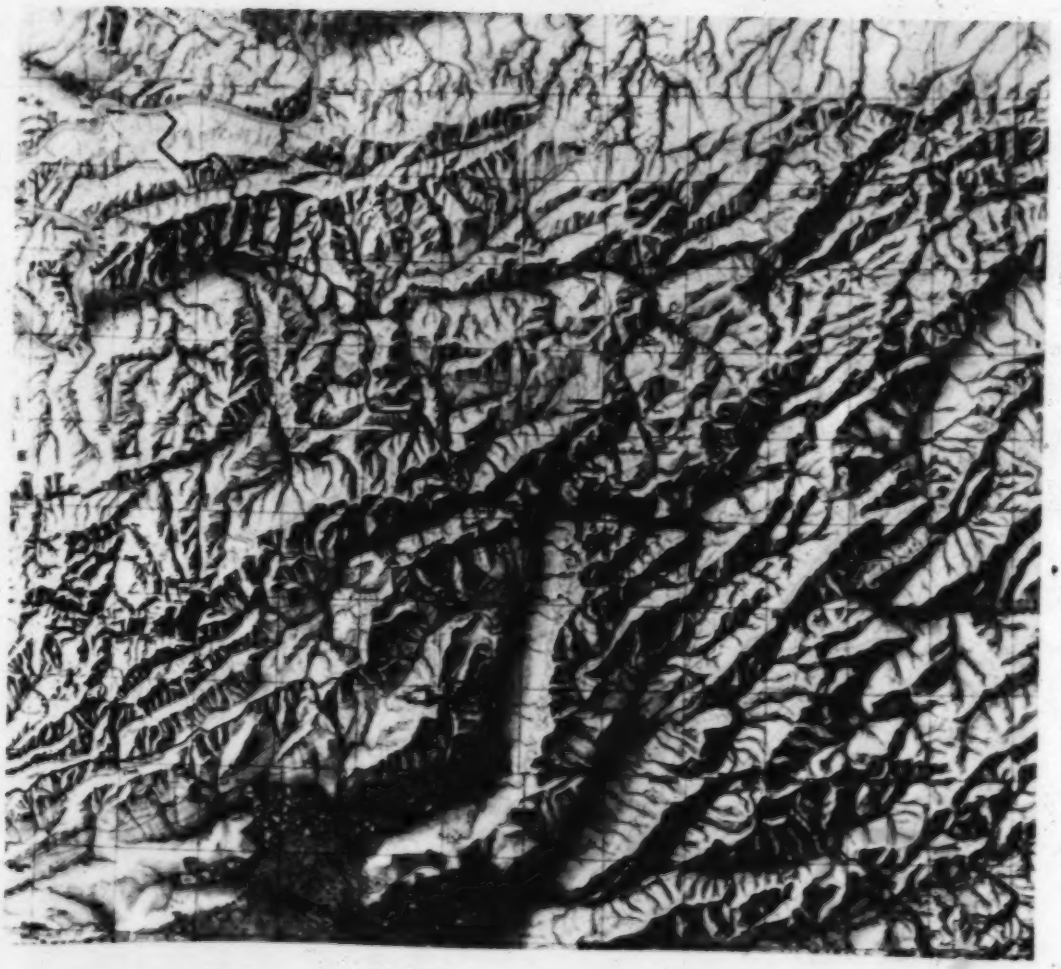

Fig. 2-6 TAPS Haul Road; Livengood to the Yukon River, Alaska (Road crosses northeast-southwest trending ridges at right angles)

of 1,000 to 1,500 feet. This entire fifty-five mile section lies within the discontinuous permafrost zone (Ferrians et al, 1969). The unglaciated surficial deposits consist of reworked aeolian silt, colluvial and alluvial silt, sand and rock fragments, dune sand, and alluvial sand and gravel and exhibits an active layer depth that varies from 18 in. to 10 ft. below the surface. These surficial sediments are unconsolidated, usually frozen, and are often very rich in ice. Ice forms include wedges, lenses, and interstitial ice.

The TAPS Haul Road has provided the first real opportunity to observe and record the performance of a significant number of road cuts in ice-rich fine grained soils. The road has many substantial cuts on north facing slopes that exposed what was termed "massive ice". Certainly the ice did appear massive in that cuts in the order of 20 ft. had 12 ft. or more of ice exposed in the section. However, the tabular extent of these ice exposures was not and has not been established.

Many of these cuts were excavated with vertical or near vertical faces with the thought that prior to any self-healing, the cuts would have to melt back to a vertical scarp. It was anticipated that the upper mantle of soil and organic mat would then slough down over the melting ice and soil and provide sufficient insulation to stabilize the slope. Vegetation would then re-establish itself preventing erosion and any further fouling of the ditches.

Inspection of these cuts in the summer of 1973 indicated that the back slopes had essentially stabilized. Many slope angles were very steep (35^{0} to 40^{0}) and the present conditions probably do not represent ultimate stable slope angles for most of the colluvial silt encountered along the route. The relatively free-draining nature

of the silt deposits, the extensive re-vegetation program, and probably the initial vertical cross section have all contributed to rapid stabilization. More important, however, may be that the ground ice occurs in the form of wedges of limited extent and that the back slopes stabilized when the exposed ice wedges melted and allowed the remaining soil to slough down forming a quasi stable slope from material with moderately low moisture contents. Summaries of the soil classification for the TAPS Haul Road appear in Table 2-1.

2.6.3 Copper River Basin

The Copper River Basin is located in southeastern Alaska in the discontinuous permafrost zone. This region has been subjected to extensive glaciation and pro-glacial lakes were common during the retreat of the ice. As a consequence, major deposits of glaciolacustrine silts and clays, fluvial silt and sand, sand and gravel, colluvium, and organic soil comprise the surficial materials. Ground ice forms and amounts are extremely variable. Ice wedges, massive tabular bodies, and interstitial ice are common. Natural water contents often exceed the liquid limit (EISTAPS, 1972).

Instability of large cuts and fills is common in this region. Slopes are designed to ensure stability in the thawed but essentially undrained case, with no excess pore water pressure considered due to thaw consolidation.

Several cuts were observed at various locations in the area. The back slopes did not exhibit any serious distress due to permafrost degradation. Many slopes along the valley wall undergo significant movements. Most of these failures are semi-rotational or block-type movements and are not necessarily related to changes in permafrost conditions.

2.7 Conclusions

The observations made during the field reconnaissance program are summarized in Table 2-2 and indicate that slope behavior varies widely in the areas that were investigated. It is also apparent that at least limited instability may be encountered in cuts in all regions which contain fine-grained, ice-rich sediments. There appear to be a number of factors which contribute to the amount of instability. These are:

(1) geological history (primarily glacial);

(2) landform and soil type;

(3) ice content of sediments;

(4) nature or type of ground ice;

(5) geometric cross-section of the back slopes.

All of these factors are inter-related and are listed in approximate order of importance.

The determination of the geological history along with the recognition of associated landforms will provide a sound basis for the preliminary design and predicted behavior of highway back slopes. Although local exceptions may exist, it appears that the greatest risk of precipitating unstable conditions arises in areas that contain significant deposits of stratified drift in the form of pro-glacial, lacustrine silts, and clays. These deposits generally contain large amounts of segregated ground ice in the form of lenses varying in thickness from a few tenths of an inch to massive sheets many feet thick (Mackay and Black, 1973).

The areal extent of these lacustrine deposits is often quite significant, varying from a few hundred feet to a number of miles. Thus, a flow slide initiated in such an extensive deposit is not likely to

terminate due to a lack of material. When these soils are thawed, large amounts of meltwater coupled with low permeabilities provide an ideal condition for the generation and maintenance of large excess pore water pressures. The thawed soil is very weak and flows at very low angles (McRoberts and Morgenstern, 1973). It is unlikely that vertical cuts would provide any great degree of stabilizing influence unless they were less than ten feet in height. Under these conditions, the intact organic cover may be able to provide a retarding effect on the rate of thaw and the subsequent generation of excess pore water pressures.

The effectiveness of the organic mat is directly related to its continuity. Tearing or ripping of the cover under self-weight can be reduced by removing the larger vegetation and in some cases reinforcing the mat with wire mesh fastened to the tree stumps.

The reconnaissance did not encounter highway or right-of-way locations in regions of extensive glacio-lacustrine deposits with the exception of the Copper River Basin in Alaska. This latter area did not exhibit the bi-modal flow slides characteristic of the Mackenzie Valley (McRoberts and Morgenstern, 1973). Although it would appear that both areas were subjected to similar glacial histories, there is a distinct difference in the ice contents of the sediments. The reasons for this deserve further study.

Conventional slope analyses using thawed, undrained strength parameters appear to provide acceptable margins of stability for most highway cuts in this region. However, there seems little doubt that route locations encountering extensive areas of pro-glacial lake basin sediments are exposed to the hazards of backslope instability. Great care must therefore be taken to avoid cuts in these regions whenever possible.

The other major classification of glacial sediments is that of unstratified drift. Till plains formed from basal or lodgement till and hummocky or dead ice moraine composed of ablation till are the two most common landforms.

Ground ice conditions in the till plains are usually not particularly severe. The ice occurs as thin subhorizontal segregated lenses. The top layer of this landform may occasionally contain significant ice lenses just below the active layer due to aggrading permafrost conditions (Hughes, 1974). These ground ice deposits usually occur within three to six feet below the surface and should, therefore, not create any sustained construction hazard. It should also be noted that this terrain has rather low relief and cuts will be uncommon.

Hummocky or dead ice moraine is common in the northern part of the Mackenzie Valley and appears sporadically along the Alaska Highway and the lower and upper portions of the Dempster Highway. This terrain unit is highly irregular and may contain lenses of silt, sand, and gravel as was witnessed at Mile 222 on the Alaska Highway and at Mile 343 on the Dempster Highway. Complex hydrologic, geologic, and topographic conditions prior to deposition of the sediments and aggradation of permafrost have produced equally complex and irregular patterns of ground ice. These forms may consist of lenses of varying thickness, wedges, and occasionally significant amounts of interstiial ice (ibid). However, it must be emphasized that these ground ice forms are usually of finite dimensions and that flow slides initiated in these sediments will be of limited (and often acceptable) extent.

Good construction practice involving wide ditches, low rock revetments, occasionally combined with minor amounts of backslope

reshaping and revegetation should provide an acceptable measure of stability and pleasing appearance.

The unglaciated regions of the Yukon and Alaska contain a variety of classic landforms consistent with the geological history and periglacial conditions. The soils are generally formed from weathered sandstones and shales and consist of colluvial and alluvial silts, sands, and gravels. Occasional deposits of medium to highly plastic residual clays are encountered in the central and northern Yukon. The ground ice conditions are again very irregular in all these deposits and occur in a variety of forms. Ice wedges and lenses of variable thickness are the most common type encountered (Kachadoorian, 1971).

The relatively high permeability of these materials coupled with low interstitical ice contents provide conditions for moderately fast stabilization under thawing conditions. Flow slides initiated in this material, with these particular ground ice conditions, will normally stabilize after two or three seasons of thaw (Davies, 1973).* Again, good construction practice should reduce the time required for stabilization and the amount of slope retreat that may be expected. Wide ditches are recommended to provide reasonable space for sloughing material. Rock revetments appear to play a significant role in retaining the soil at the toe of the backslope while drainage proceeds. Vertical cuts may be desirable in areas that contain soil and ice conditions as previously described in that they tend to expose or intersect fewer ice wedges. This aspect combined with the steep quasi-stable angles developed by the soil between and above the ice provide

* Personal communication.

encouragement for the designer in the use of steep slopes. However, it must be noted that generalization is not possible with the current state of knowledge and that steep cuts will not be desirable everywhere. There is no evidence and little rationale to suggest that vertical slopes will perform satisfactorily in ice-rich glaciolacustrine sediments except under conditions previously outlined.

The occurrence of instability of natural slopes is undoubtedly the best indicator available to assess or predict potential instability of cut slopes. If areas do not exhibit natural instability due to permafrost degradation then it is unlikely that cutting of a slope will cause serious and extensive instability in the form of flow slides.

The only reliable method encountered for preventing rather than controlling flow slides was that utilized on the Mackenzie Highway at the approach cut to the Mackenzie River near the settlement of Arctic Red River. A thick layer of crushed rock was used to dress the exposed slope before degradation of the permafrost occurred.

Procedures to prevent melting of the permafrost in colder regions are well known and are relatively simple in comparison with preventing permafrost degradation in the discontinuous zone (Lachenbruch, 1957).

The experience with cut slopes in permafrost reviewed here, albeit limited, indicates that the behavior of cuts need not be a serious impediment to the routing of transportation arteries across permafrost.

TABLE 2-1

SUMMARY OF SOIL DATA

LOCATION	SOIL TYPE	LIQUID LIMIT %	PLASTIC LIMIT %	NATURAL WATER CONTENT %	SPECIFIC GRAVITY	SAND AND GRAVEL %	SILT %	CLAY %
Alaska Highway Mile 222	Silt			23		30	60	10
Alaska Highway Mile 222	Silty-clay	50	19			10	50	40
Dempster Highway Mile 108	Silt					20	75	5
Dempster Highway Mile 114	Silt	27		25	2.71	10	80	10
Dempster Highway Mile 153	Silt					20	75	5
Dempster Highway Mile 174		54	25				40	60
Dempster Highway Mile 175		45	23	30-40		5	35	60
Dempster Highway Mile 175		48	24	30-40		5	45	50
Bi-modal Flow Richardson Mts.	Till	36	22	Dry	2.70	30	40	30
Bi-modal Flow Richardson Mts.	Alluvium	39	24	38	2.72	15	53	32
TAPS Haul Road Mile 19.6	Silt	25				0	92	8
TAPS Haul Road Mile 23.4	Silt					10	83	7
TAPS Haul Road Mile 54.4	Silt	24				0	91	9
Dempster Highway Mile 343	Upper till	26	18			25	55	20
Dempster Highway Mile 343	Lower till	23	15			30	40	30

TABLE 2-2

SUMMARY OF CUTS

LOCATION	HEIGHT (Feet)	INITIAL SLOPE (Degrees)	CURRENT SLOPE (Degrees)	SOIL TYPE	COMMENTS
Alaska Highway Mile 222	20-25	26.5	Varies from near vertical to 6° frontal lobe.	Till with lenses of silt and silty clay.	The slope had retreated approximately 15 feet at the time of inspection and gave no indication of stabilizing.
Dempster Highway Mile 96	8-10	26.5	21	Till-relatively fine-grained with low ice content.	Slope is stable.
Mile 107.6	30	34	32	Colluvial silt and weathered shale with low ice content.	Organic mat tearing loose; slope is stable.
Mile 108.9	35	34	12 to 35	Colluvial silt, some weathered shale. Large ice lenses.	Slope has stabilized with small amounts of ditch maintenance.
Mile 114 Tote Road	Instability of natural slope		9 to 11	Alluvial silt and slope wash. High ice content.	Caused by locating tote road on 5° side hill. Disturbance approximately 6 years old and continuing to stabilize.
Mile 120.7	Less than 10	26.5	12	Ice-rich organic alluvial and/or colluvial silts.	Slope presently quite stable. Some ditch clearing required to maintain drainage.

TABLE 2-2 (Continued)

LOCATION	HEIGHT (Feet)	INITIAL SLOPE (Degrees)	CURRENT SLOPE (Degrees)	SOIL TYPE	COMMENTS
Trans-Alaska Pipeline System Haul Road Mile 2	40	26.5	26.5	Weathered sandstone bedrock.	Slope is stable and revegetation is well established.
Mile 14	10 to 25	26.5	Backscarps at 35 to 45; lobes at 7 to 10.	Colluvial silt and massive ice.	Trees were removed by hand and the moss has draped itself over the retreating face. The slope is essentially stable.
Mile 19.6	15 to 20	34	20	Colluvial silt with large ice lenses and wedges.	Crushed rock revetments were used to retain the sloughed material.
Mile 20.3 to Mile 20.5	20 to 40	76	35 to 45	Colluvial silt with large ice lenses and wedges.	This section was referred to as the "ice tunnel". Vegetation is well established.
Mile 22.9	15 to 20	76	Varies from 17 to 40	4 to 6 feet of organic matter over silt and ice.	Boreholes indicated massive ice in excess of 32 feet. Slope degraded 75 feet in localized areas.

TABLE 2-2 (Continued)

LOCATION	HEIGHT (Feet)	INITIAL SLOPE (Degrees)	CURRENT SLOPE (Degrees)	SOIL TYPE	COMMENTS
Mile 153	40	40 to 45	Angle of repose of coarse gravel	80% silt; 20% shale; no interstitial ice. Large lenses or wedges exposed.	Backslope retreated until a 35 foot vertical section was formed. Gravel placed on face to control melting. Slope appears stable.
Mile 168	Less than 10	34	11	Sandy silt with angular weathered rock. Ice-rich.	Backslope has retreated 20 to 25 feet. Appears to be stabilizing under present conditions.
Mile 175	8 to 10	34	40 to 50	Silt and silty clay with shale fragments.	Activity due to melting of large ice lense, slope has retreated 25 feet at time of inspection.
Mile 343	35	34	17	Silty clay till; ice lenses up to 4 feet in thickness.	Cut is 1/2 mile long. Backslopes are essentially stable; some ditch clearing and backslope reshaping has been done.
Mackenzie Highway Mile 894	45	34 on upper slope to 45 on lower slope	45	Upper 25 feet is ice-rich and fine-grained Lower 20 feet is bedrock.	Upper 25 feet was benched and dressed with 10 foot thick layer crushed shale slope is completely stable.

CHAPTER III

STABILIZATION OF PLANAR LANDSLIDES IN PERMAFROST

3.1 Introduction

Flow dominated land slides in periglacial regions have been documented by numerous researchers (Capps, 1919; Eakin, 1919; Sigafoos and Hopkins, 1952; and others). Skin flows or planar landslides involving the detachment of vegetation and thawed soil from the underlying frozen soil are abundant in the Mackenzie Valley and have been described by Hardy and Morrison (1972), Hughes (1972), Issacs and Code (1972), Mackay and Mathews (1973), and McRoberts and Morgenstern (1973).

Since skin flows are common, a method of stabilizing them or inhibiting their occurrence is of interest.

A technique for the analysis of thawing slopes of infinite extent has been developed which incorporates pore water pressure components due to steady state seepage as well as those resulting from self weight and surcharge loading during thaw consolidation. Insulation used in conjunction with a surcharge load reduces the rate of thaw, decreases the induced pore water pressures and may effectively stabilize the slope.

3.2 Thaw-Consolidation Pore Water Pressures

Excess pore pressures in thawing soils may be predicted from the theory given by Morgenstern and Nixon (1971). The magnitude of these water pressures at a horizontal thaw front in a homogeneous single layer soil due to self weight and surcharge loading are given by the following equations:

$$U_{\gamma}' = \gamma' \; X(t) \frac{1}{1 + \frac{1}{2R^2}} \quad - - - [3.1]$$

$$U_{p_o} = \frac{(P_o - \sigma_o') \, erf(R)}{erf(R) + \frac{e^{-R^2}}{\sqrt{\pi R}}} \quad - - - [3.2]$$

where γ' = effective unit weight of the soil

$X(t)$ = depth of the thaw front, predicted by

$X(t) = \alpha \sqrt{t}$ (Carslaw & Jaeger, 1947) - - - [3.3]

where $\alpha = \sqrt{\frac{2K_u Ts}{L}}$ (Stefan, 1891) - - - [3.4]

K_u = thermal conductivity of the unfrozen soil;

Ts = surface step temperature;

L = latent heat of fusion of the soil/ice mixture;

t = time

$R = \frac{\alpha}{2\sqrt{C_v}}$ (thaw consolidation ratio) - - - [3.5]

erf ( ) = error function

P_o = surcharge load

σ_o' = residual effective stress (assumed to be zero at shallow depths in ice rich fine grained soils). The solution for equations [3.1] and [3.2] have been given in graphical form by Morgenstern and Nixon (1971).

The depth of thaw in a two layer medium is given by Nixon and McRoberts (1973):

$$X(t) = \sqrt{\left(\frac{K_2}{K_1} H\right)^2 + \frac{2K_2 \, Ts(t - t_o)}{L_2}} - \left(\frac{K_2}{K_1} H\right) \quad - - - [3.6]$$

where $X(t)$ = depth to the thaw front measured from the interface of the two layers.

K_1 = thermal conductivity of the upper layer

K_2 = thermal conductivity of the lower layer

H = thickness of the upper layer

t = length of the thawing season

$t_o = \frac{H^2 L_1}{2K_1 \Gamma_s}$; (time to thaw layer 1)

L_2 = latent heat of fusion of the soil/ice mixture in the lower layer.

$$L_{(1,2)} = \gamma_t (W - W_u) L' \quad - - - [3.7]$$

where γ_t = total unit weight of the soil

W = water content of the soil

W_u = unfrozen water content of the soil expressed as a ratio of gms. of water to gms. of dry soil

L' = latent heat of fusion of ice (79.6 cal/gm.)

If the upper layer consists of an insulating material or incompressible surcharge loading, a measure of the pore pressure in the soil below the insulation may be determined by computing $X(t)$ from equation [3.6], substituting into equation [3.3] and obtaining an equivalent α_e. R can then be evaluated from equation [3.5] and the pore pressures evaluated for this modified rate of thaw.

The above procedure is not strictly correct because the thaw depth as computed from equation [3.6] will in general not be linear with the square root of time. Figure 3.1 indicates the manner in which the thaw depth varies with time for a typical problem. It is apparent that this technique will slightly underestimate the thaw-

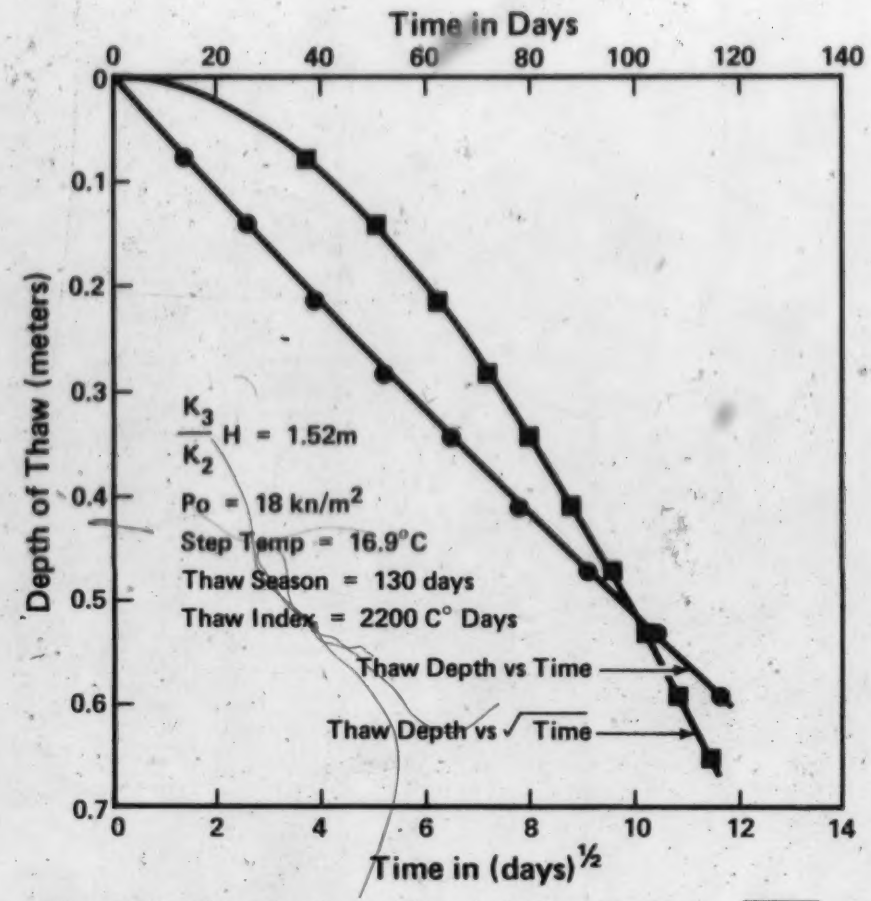

Fig. 3.1 Thaw Depth vs Time and Thaw Depth vs $\sqrt{\text{Time}}$ for an Insulated Thawing Slope

consolidation pore pressures generated in this particular two layer medium. A better estimate of these water pressures can be made either by numerical techniques or by modelling the thaw rate from:

$$X(t) = B(t)^n \quad \text{(Nixon and Morgenstern, 1973)} \quad - - - [3.8].$$

The thaw depth must be calculated for various times from equation [3.6] and a suitable value of n determined for equation [3.8]. Graphical procedures may then be used to properly evaluate the pore pressures at the thaw front at different periods of time (ibid).

3.3 Stability Analysis of Infinite Thawing Slopes

The pore pressure term in the stability analysis of an infinite slope with surcharge loading will be composed of:

1) Pore pressure due to steady state seepage.
2) Thaw-consolidation pore water pressure due to self weight.
3) Thaw-consolidation pore water pressure due to surcharge loading.

These three components may be evaluated separately as shown in Figure 3.2 and then added numerically to obtain the total pore water pressure acting at the thaw front. This procedure is similar to that of McRoberts (1972), and McRoberts and Morgenstern (1974).

The condition for steady state seepage is shown in Figure 3.2(a). This is a well known solution and the pore pressure term is:

$$U_s = X(t) \cos i \, \gamma_\omega \quad - - - [3.9(a)]$$

Figure 3.2(b) indicates the condition for the pore pressure due to self weight generated from one dimensional thaw-consolidation. This magnitude is expressed by:

$$U_{\gamma'} = \gamma_3' \, X(t) \cos i \, \frac{1}{1 + \frac{1}{2R^2}} \quad - - - [3.9(b)]$$

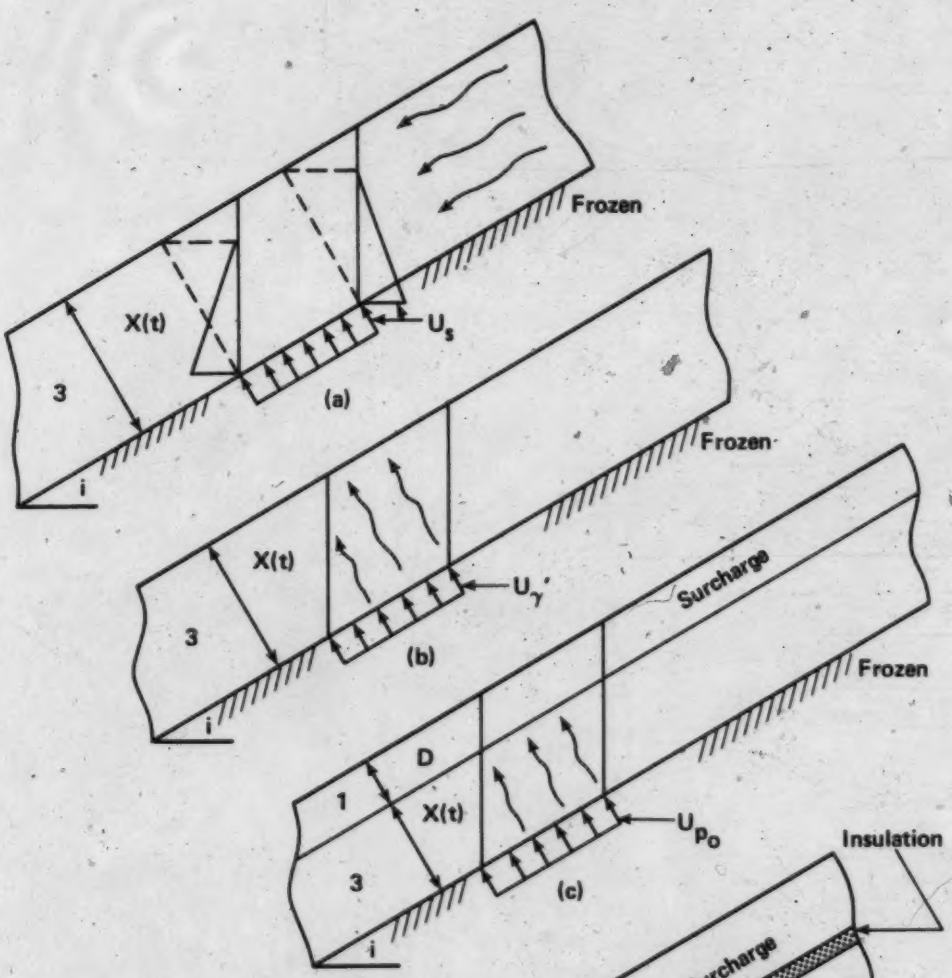

Fig. 3.2 Pore Pressure at the Thaw Front of a Thawing Infinite Slope

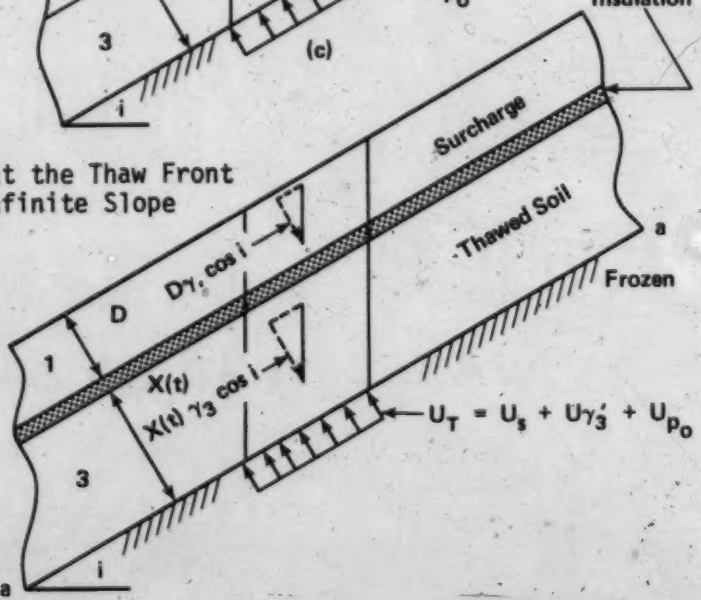

Fig. 3.3 Balance of Forces on a Thawing Infinite Slope

where γ_3' X(t) cos i is the normal stress on the thaw plane produced by the effective unit weight of the thawed soil. The pore pressure due to the surcharge is shown in Figure 3.2(c) and is given by:

$$U_{P_o} = \frac{P_o \cos i \; erf(R)}{erf(R) + \frac{e^{-R^2}}{\sqrt{\pi R}}} \quad - - - [3.9(c)]$$

where P_o cos i is the normal pressure exerted by the surcharge on the thaw plane.

If $R' = \frac{1}{1 + \frac{1}{2R^2}}$ and

$$R'' = \frac{erf(R)}{erf(R) + \frac{e^{-R^2}}{\sqrt{\pi R}}}$$ then

$$U_T = X(t) \cos i \; \gamma_\omega + \gamma_3' X(t) \cos i \; R' + P_o \cos i \; R'' \quad - - - [3.9(d)]$$

R' and R'' may be evaluated from Figure 3.4 and 3.5 respectively. A statical balance of forces applied at the thaw front as indicated by section a-a in Figure 3.3 results in a factor of safety:

$$F = \frac{\{D\gamma_1 (1 - R'') + \gamma_3' X(t) (1 - R')\}}{D\gamma_1 + \gamma_3 X(t)} \frac{\tan\phi'}{\tan i} \quad - - - [3.10]$$

where γ_1 and γ_3 = total unit weights of the surcharge and the thawed soil respectively.

γ_3' = effective unit weight of the thawed soil.

i = angle of inclination of the slope.

If the depth or unit weight of the surcharge is zero, the equation becomes:

$$F = \frac{\gamma_3'}{\gamma_3} (1 - R') \frac{\tan\phi'}{\tan i} \quad - - - [3.11]$$

which is in agreement with McRoberts (1972) and McRoberts and Morgen-

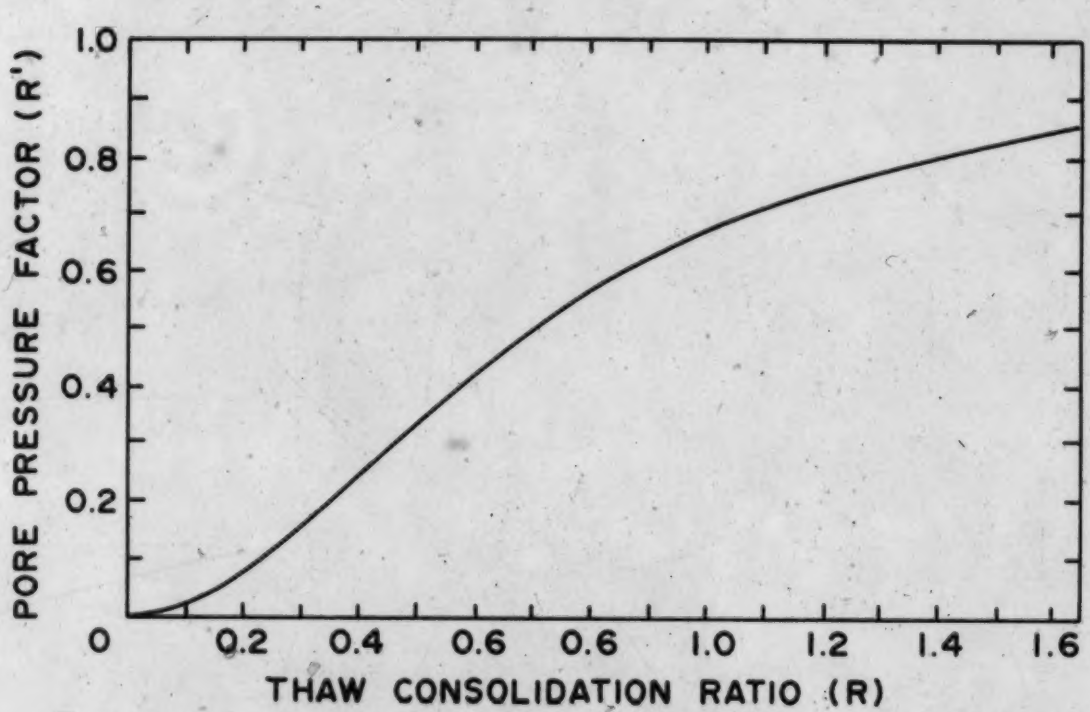

Fig. 3.4 Pore Pressure Factor (R') (self weight only) for Different Thaw Consolidation Ratios (R). After Morgenstern & Nixon (1971).

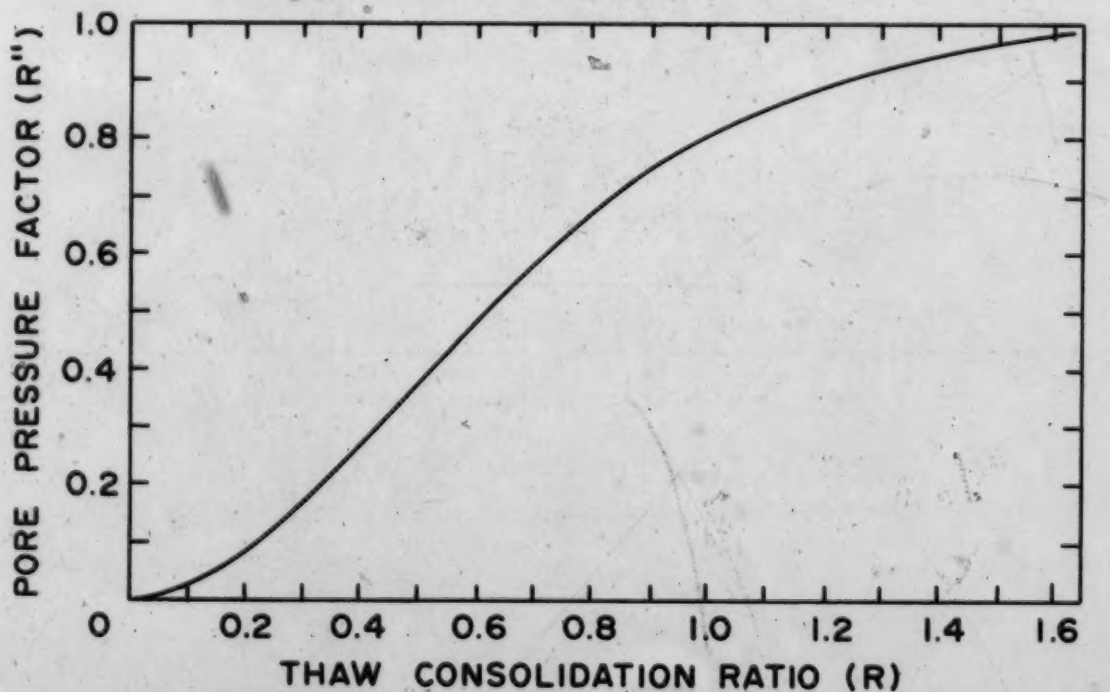

Fig. 3.5 Pore Pressure Factor (R") (surcharge only) for Different Thaw Consolidation Ratios (R). After Morgenstern & Nixon (1971).

stern (1974). Equation [3.10] produces an approximate and slightly unconservative solution because the procedure underestimates the pore water pressures and therefore overestimates the factor of safety. If the rate of thaw is plotted as in Figure 3.1 and modelled from equation [3.8], R' and R'' as obtained from Nixon and Morgenstern (1973) will be larger and the factor of safety will be proportionately smaller. The relative magnitude of these safety factors will be compared later.

3.4 Numerical Procedure For Design

The need for a numerical procedure to analyze the stability of a thawing slope covered with insulation and surcharge load is based on the lack of a suitable analytical method to evaluate pore water pressures for rates of thaw that are not proportional to the square root of time. A program was developed to compute the depth of thaw from equation [3.6] and uses a Crank-Nicholson finite difference procedure to solve the equation of consolidation subject to the boundary conditions at the thaw line (Nixon, 1973). The factor of safety is computed from:

$$F = \left[\frac{D\gamma_1 + \gamma_3' X(t) - U(N)/\cos i}{D\gamma_1 + \gamma_3 X(t)}\right] \frac{\tan \phi'}{\tan i} \quad - - - [3.12]$$

where U(N) is the pore pressure due to thaw consolidation and surcharge loading acting at the thaw front, and the other terms are as defined previously. This equation is identical to equation [3.10], except for the pore pressure term (U(N)).

The input parameters are:

1) thermal conductivities of layers 1, 2 and 3 (cal/C^{o} cm sec) (Kersten, 1949 and Table 3.1);

TABLE 3-1

PHYSICAL AND THERMAL PROPERTIES OF VARIOUS INSULATING MATERIALS

Material	Moisture Content %	Density gm/cc	Specific Heat cal/gm ^{o}C	Thermal Conductivity cal/^{o}c cm sec
1) *Styrofoam HI	Dry	0.052	0.28	7.9×10^{-5}
2) *Polyurethane	Dry	0.03 to 0.08	0.28	5.5×10^{-5}
3) *Fibreglass	Dry	0.048	0.16	8.2×10^{-5}
2) *Woodchips	37	0.26	0.33	3×10^{-4}
3) *Sawdust or shavings	Dry	0.19	0.33	1.6×10^{-4}
4) *Peat	Varies with water content			

1)* Dow Chemical (1972)

2)* Smith, Berg and Muller (1973)

3)* ASHRAE (1963)

4)* Dept. of Army (1966)

2) trial thickness of insulation (m);
3) the equivalent step temperature (^{o}C);
4) the length of the thawing season (days);
5) the coefficient of consolidation of the parent material (cm^2/sec);
6) the grid spacing in the parent material (a suitable depth increment is = 0.040 m);
7) the thickness and unit weight of the surcharge respectively (m and kN/m^3);
8) the specific gravity of the soil solids in the parent material;
9) the total and unfrozen water contents of the parent material (expressed as a ratio of gms of water/gms of dry soil);
10) the effective angle of internal friction of the parent material;
11) the angle of the slope with the horizontal (^{o}); and,
12) the number of times output is desired over the length of the thaw season.

The output information consists of elapsed time, the depth of thaw, the pore pressure depth increment profile, the effective normal and shearing stress at the thaw front, and the factor of safety against failure.

If pore pressures and corresponding factors of safety are to be computed throughout the thaw season, then an equivalent surface step temperature must be used in equation [3.6]. This temperature is a function of the air thawing index (Thompson, 1963), a surface index factor (N) (Dept. of U.S. Army, 1966), and the length of the thaw

season (Burns, 1973).

Thus $$Ts = \frac{ATI \times N}{\text{Length of thaw season}} \qquad - - - [3.13]$$

On the other hand, if pore pressures and factor of safety are desired only at the end of the thaw season when the thaw depth is at its maximum value and the factor of safety is at its minimum value, then only the surface thawing index need be used in equation [3.6]. The surface thawing index is the product of the surface temperature Ts and the total length of the thaw season. Generally, the time (t_o) required to thaw through the gravel surcharge will not be significant and may, therefore, be ignored. This simplifies the computations and provides a more conservative answer. The number of days that are chosen to represent the length of the thaw season is not particularly critical, for it is the product of the length of the thaw season and the step temperature which governs the maximum depth of thaw and is, therefore, of interest in the final analysis. It is, however, conservative to choose a slightly shorter thaw season and a correspondingly larger step temperature. Under these conditions larger pore pressures are generated in the early stages of the thaw season and therefore accentuate any critical condition that may occur at that time.

The actual temperature boundary conditions are approximated more closely by a sinusoidal variation rather than a surface step temperature. It is of interest to note that this former temperature variation will produce larger rates of thaw and therefore larger pore water pressures in mid-season than does an equivalent step temperature boundary condition (Nixon and McRoberts, 1973). At late stages in the thaw season the velocity of the thaw front is

reduced and eventually goes to zero when the thaw depth reaches its maximum value. Although the factor of safety of the slope decreases with increasing thaw depth, it is unlikely that the increase in the velocity of the thaw front at mid-season combined with a relatively shallow depth of thaw would seriously decrease the factor of safety at that time. On the other hand, the reduction in the velocity of the thaw front at the end of the thaw season diminishes the severity of the condition of decreasing factor of safety with increases in depth of thaw.

Estimates of thermal conductivity for the surcharge (K_1) and in situ material (K_3) may be obtained from Kersten (1949). These values must be multiplied by 4.13×10^{-3} to change the units to cal/cm ^{o}C sec. Table 3.1 lists the thermal conductivities (K_2) of various insulation materials that may be used. The surcharge material will provide some small but significant insulative value. The U.S. Army and Air Force (1966) suggest that one layer of material may be converted to an equivalent thickness of another from the ratio of their diffusivities. If the simple Stefan solution is employed to calculate the rate of thaw, then the ratio of the conductivities may provide a reasonable and conservative evaluation of the insulating effect of the sand/gravel surcharge. It should also be noted that the latent heat effects of the surcharge and insulation have been omitted from the computations. This term acts to retard the rate of thaw through these upper layers and therefore essentially reduces the length of the thaw season. On the other hand, the large water contents associated with significant magnitudes of latent heat increase the thermal conductivity of these materials and reduce their insulating

value. It was, therefore, considered preferable to neglect the latent heat effects of the upper layers and assume that the time to thaw through these upper two layers (to) in equation [3.6] be equal to zero.

Values of the coefficient of consolidation (Cv) may vary from 1×10^{-2} to 5×10^{-2} cm.^{2}/sec. for materials encountered in the active layer at a number of sites in the Mackenzie Valley (Roggensack, 1975).* Test results also indicate that Cv will decrease with depth to values in the order of 3×10^{-3} cm.^{2}/sec. when tested at pressures equal to or greater than the effective overburden. The minimum value that may be encountered will not likely be less than 5×10^{-4} cm.^{2}/sec. (ibid). However, when cuts are made in this material, the soil will be thawing under low normal stresses and the tendency for consolidation will be minimal. The values of Cv quoted for the active layer are therefore considered to be appropriate for the majority of the situations encountered in this region of the Arctic.

Values of ϕ' will vary from 20 to 24 degrees for the predominantly illitic clays found in this region (ibid). The effective cohesion intercept has been considered as zero for the purposes of computation.

3.5 Design Charts

A series of design charts have been prepared for use in determining the appropriate thicknesses of insulation and surcharge that may be required to stabilize thawing slopes in frozen soils (Figures 3.6 to 3.17). These charts establish the relationship between the factor of safety of a thawing slope and the factor of

---

* Personal communication

safety of the same unfrozen slope with steady state seepage pore pressures only. The ratio F thaw/F seepage is plotted against the coefficient of consolidation. Four curves appear on each plot indicating conditions for surcharge loads varying from 0 to 18.0 kN/m^2 (0 to 375 psf).

Three surface thawing indices, (1,500; 2,500 and 3,500 C^o days) were chosen to embrace thawing conditions extending from the latitudes of Fort Simpson to the Beaufort Sea. Four water contents varying from 20 to 50% span the range likely to be encountered for most problems. The ratio of K_3/K_2 H indicates the effective insulation thickness. The three values of 0, 1.52 m (5.0 ft.) and 3.05 m (10.0 ft.) provide the necessary range for typical conditions.

The factor of safety varies with time, decreasing with increasing time and thaw depth. Therefore, only the minimum values that correspond to the maximum thaw depths have been considered.

The total unit weight of the surcharge was taken as 19.6 kN/m^3 (125 pcf) with a water content of 12% and a thermal conductivity (K_1) of 0.0065 cal/^oC cm. sec. The permafrost was assumed to be saturated and the soil grains to have a specific gravity of 2.70. Thaw depths were computed using unfrozen thermal conductivities (K_3) that vary with water content according to Kersten (1949). The unfrozen water content was assumed to be zero.

3.6 Example Problem

The information for the example problem regarding soil, insulation, climate, and desired factor of safety appears in Table 3-2. The solution is obtained by referring to Figures 3-6 to 3-17.

Trial I determines the thaw depth and the F ratio for the

TABLE 3-2

SOIL, INSULATION AND CLIMATE INFORMATION FOR EXAMPLE PROBLEM

SOIL PROPERTIES	INSULATION PROPERTIES	CLIMATE	F_1 (SEEPAGE) $\frac{\gamma_3}{\gamma_3} \frac{\tan \phi'}{\tan i}$	F_R
W = 35%	wood chips:	Air thawing Index: 1260 C^o days (Thompson, 1963)	0.89	$\frac{1.3}{0.89} = 1.46$
Wu = 0	K_2 = 0.003 cal/^{o}C cm sec			
Cv = 0.002 cm^2/s	Styrofoam:	Surface Thawing Index Factor: 1.75 (U.S. Dept of Army, 1966)		
ϕ' = 22^o	K_2 = 0.000079 cal/^{o}C cm sec			
K_3 = 0.003 cal/^{o}C cm sec		Surface Thawing Index: 1260 x 1.75 = 2200 C^o days		
Slope Angle = 12^o				

* Assume F (Thaw) required = 1.3

slope after disturbance, but without insulation or surcharge. The results are obtained by linear interpolation between soil water contents of 30 and 40% and surface thawing indices of 1,500 and 2,500 C^{o} days.

Trial II determines the thaw depth and F ratio for a $\frac{K_3}{K_2} H$ ratio of 1.52 m (5.0 ft.) and a surcharge load of 18.0 kN/m^2 (375 psf) (1 m of surcharge) for the same water content and surface thawing index. An F ratio of 1.47 was obtained for this combination of insulation and surcharge.

Trial III produces results for a $\frac{K_3}{K_2} H$ value of 2.3 m (7.5 ft.) and a surcharge load of 12 kN/m^2 (250 psf) (≈ 0.6 m of sand/gravel). An adequate F ratio is obtained with this combination of surcharge and insulation.

Linear interpolation for thaw depth between thawing indicies produces negligible error. However, linear interpolation between $\frac{K_3}{K_2} H$ values of 0 and 1.52 m causes a maximum of 8% overestimation of the thaw depth. Overestimation of thaw depth between $\frac{K_3}{K_2} H$ values of 1.52 m and 5.05 m is not significant.

Both Trial II and Trial III solutions are viable but it is probable that Trial II would be more economical depending on the relative availability of surcharge and insulating materials. However, the thaw depth in Trial II is approximately 20% greater than in Trial III. This may be a distinct disadvantage if the lower material is significantly inferior to the surficial deposits. Trial II would require approximately 15 cm. (6 in.) of woodchips along with 1 m (3 ft.) of sand/gravel surcharge. If styrofoam board were used approx-

imately 4 cm. (= 1.51 in.) would provide the same $\frac{K_3}{K_2}$ H value.

If the same problem is solved by direct hand computations, the thaw depth is computed from equation [3.3]. The modified thickness of insulation is determined from:

$$HE = H + \frac{K_2}{K_1} DEP \qquad - - - [3.14]$$

where H = thickness of insulation

DEP = thickness of sand/gravel surcharge

K_1 and K_2 are the thermal conductivities of the surcharge and insulation respectively. α_e is determined from equation [3.3], R from equation [3.5], R' and R" from Figures 3.3 and 3.4 and F from equation [3.10]. The results are shown in Table 3-3.

Figure 3-1 illustrates the manner in which the thaw depth varies with time for the combination of insulation and surcharge represented by Trial II. The thaw depth is almost linear with time, indicating a reasonable value of n (in equation [3.8]) equal to 1. R' and R" may be evaluated from Nixon and Morgenstern (1973) and F computed from equation [3.10]. The results appear in Table 3-4.

A comparison of the values for factor of safety in this example indicate a slight variance in the magnitude depending on the method chosen for evaluation. The maximum value of the difference in this example is less than 10%. Since it is impossible to predict the various soil and climatic parameters required for the solution within this tolerance, it may be concluded that all methods produce acceptable values for design. The design charts are favoured in a choice of methods because of their convenience. The numerical procedure provides a more exact solution and generates data throughout

TABLE 3-3

THAW DEPTH AND INSULATION AND SURCHARGE REQUIREMENTS FOR EXAMPLE PROBLEM

Surface Thawing Index C^o Days	Water Content %	$\frac{K_3}{K_2}$ H (m)	Po (kN/m^2)	Thaw Depth (m)	F_R
TRIAL I					
1500	30	0	0	1.57	0.64
1500	40	↓	↓	1.33	0.71
1500	35	↓	↓	1.45	0.68 ←
2500	30	↓	↓	2.03	0.54
2500	40	↓	↓	1.71	0.63
2500	35	↓	↓	1.87	0.59 ←
2200	35	0	0	1.74	0.62 ←
TRIAL II					
1500	30	1.52	18	0.53	1.50
1500	40	↓	↓	0.38	1.77
1500	35	↓	↓	0.45	1.63 ←
2500	30	↓	↓	0.82	1.25
2500	40	↓	↓	0.61	1.54
2500	35	↓	↓	0.71	1.40 ←
2200	35	1.52	18	0.64	1.47 ←
TRIAL III					
1500	30	1.52	12	0.57	1.37
1500	40	↓	↓	0.42	1.63
1500	35	↓	↓	0.49	1.50 ←
2500	30	↓	↓	0.94	1.13
2500	40	↓	↓	0.71	1.39
2500	35	↓	↓	0.82	1.26 ←
2200	35	1.52	↓	0.72	1.33 ←
1500	30	3.05	↓	0.34	1.60
1500	40	↓	↓	0.25	1.84
1500	35	↓	↓	0.30	1.72 ←
2500	30	↓	↓	0.58	1.39
2500	40	↓	↓	0.40	1.66
2500	35	↓	↓	0.49	1.53 ←
2200	35	3.05		0.43	1.59 ←
2200	35	2.3	12	0.59	1.46 ←

TABLE 3-4

COMPARISON OF FACTORS OF SAFETY BY DIFFERENT METHODS OF COMPUTATION

FOR $\frac{K_3}{K_2}$ H = 1.52 m Po = 18 kN/m^2 (TRIAL II)

$X(t) \alpha \sqrt{\text{Time}}$ Equation (3-10)	$X(t) \alpha$ Time Equation (3-8)	Design Charts	Numerical Methods
	n = 1.0		
Max. Thaw Depth 0.649 m (2.13 ft)	Max. Thaw Depth 0.649 m (2.13 ft)	Max. Thaw Depth 0.64 m (2.10 ft)	Max. Thaw Depth 0.649 m (2.13 ft)
α_e = 0.0194 cm/sec$^{1/2}$	α_e = 0.0194 cm/sec$^{1/2}$	F = 1.31	F = 1.28
R = 0.217	R' = 0.14		
R' = 0.086	R" = 0.16		
R" = 0.090	F = 1.28		
F = 1.36			

the complete thaw season. The choice between the latter two methods will, therefore, depend upon the extent of the information required in the design project.

3.7 Recommendations For Installation

Since there is little documented experience regarding the actual performance of such installations, the following section is not intended to provide a detailed contract specification, but rather to highlight some of the more important aspects of the topic. Each site will demand innovative methods depending upon location, geographical setting, accessibility and a host of other logistic problems.

The ideal time for installation of the stabilizing materials would be in late winter or early spring when the permafrost is at its minimum temperature. It is intended that the placement of the insulation precede that of the surcharge. It may, however, be necessary to dress the slope with a layer of sand and gravel to provide a uniform surface for the insulation.

It is imperative that adequate drainage be provided directly above the permafrost to prevent the possibility of impeding the dissipation of excess pore water pressures. This may be most readily achieved by placing a four to six inch granular inverted filter or perforated plastic drainage pipe embedded in a herringbone pattern within a layer of sand immediately adjacent to the base material.

Insulation in the form of styrofoam board, wood chips, sawdust or planer shavings would then be placed over this drainage facility. Some consideration might be given to prevent these insulating materials

from absorbing excessive surface run-off and thus increasing their thermal conductivities. Styrofoam board will absorb 0.25% water by volume (Dow Chemical, 1972) and this small amount of moisture retention will not measurably affect the thermal characteristics of this material. Similar data is not available for foamed in place polyurethane but it is reasonable to assume that its water retention characteristics would be somewhat greater than the extruded styrofoam board. Increases in thermal conductivity with increasing water content in planer shavings and wood chips are also not known. However, these materials may be protected with an overlay of polyethylene plastic that would help to prevent substantial increases in water content. Surface runoff should be diverted by construction of small diversion dykes above the slope rather than by ditching.

The surcharge material, consisting of a sand/gravel mixture capable of sustaining plant growth forms the upper layer of the stabilizing components. This material must be end dumped and worked down the slope using all reasonable care to prevent disturbance of the underlying materials. It is necessary to ensure that this material is sufficiently resistant to erosion to maintain it in place prior to re-establishing vegetation on the slope.

Re-vegetation procedures should take place subsequent to the final trimming of the slope. It would be desirable to apply the seed in the form of hydro seed at the beginning of the growing season. The choice of plant species should be made in accordance with the results of on-going research experiments being conducted in the arctic and sub-arctic regions.

3.8 Longterm Performance

The most critical conditions regarding pore water pressures and stability will not necessarily occur during the first season of thaw. During this period of time, the soil will be consolidating under the imposed surcharge load and the water content will be reduced accordingly. Freeze back in the fall will not occur until movement of the thaw front has ceased and the pore water pressure has essentially dissipated. Therefore, there does not appear to be any danger of impeded drainage due to freeze back from the surface. During the second season of operation the $\frac{K_3}{K_2} H$ ratio will be increased because of the increased thermal conductivity of the soil due to the decrease in water content. However, this increase is not large enough to limit the melting to the same level during the first year. As a consequence, the thaw line will be penetrating the material below the maximum thaw depth of the preceding year, resulting in a smaller factor of safety. The magnitude of the decrease is less than 10% for the extreme condition, where the water content is reduced from 50 to 20%. These changes in factor of safety from one season to another are generally not considered to be significant. However, the designer may wish to choose a slightly larger initial value for the factor of safety in view of the foregoing. Increased thaw depth during subsequent years will not be significant. As vegetation is developed on the slope the surface heat balance will change and the surface will experience cooler temperatures, resulting in a decreased thaw depth and increased stability.

3.9 Choice of Insulation

A number of different insulating materials have been listed

in Table 3-1 for consideration. The final choice of material for each location will be made on the basis of availability of the material, the ease of placement and ultimately, the cost in place. A detailed economic analysis is beyond the scope of this presentation and the following discussion is only meant to provide guide lines for the designer in his choice of material.

Styrofoam board is readily available in southern Canada and may be shipped by rail f.o.b. Hay River, N.W.T. for approximately 17 cents per board foot (1974 quotation). This material is easy to place but requires a significant amount of unskilled hand labour. It has a low value of thermal conductivity, low water retention characteristics and a high compressive strength. It is not biodegradable, which may or may not be an advantage.

Polyurethane (foamed in place) has insulative qualities similar to those of styrofoam. It requires a high level of expertise and sophisticated equipment for placement. As a comparative cost, this option has been quoted (Edmonton, 1974) at 25 cents per board foot for material and $1.00 per square foot for placement. Mobile equipment is available for transport into remote regions.

Large quantities of planer shavings are available at points as far north as High Level, Alberta. Small quantities may be available from local saw mills in the southern part of the Yukon and Northwest Territories. The cost of this material is very low. On the other hand, transportation costs may be very high. Placement is simple, requiring combined machine and hand operations. Plastic sheeting should be used to encapsulate the shavings and prevent excessive increases in the water content. This insulating material may undergo a substantial

amount of compaction under the influence of the surcharge load. Some reduction in insulative quality will undoubtably result. Placing 50-75% of the surcharge below the shavings would help to alleviate this problem. Planer shavings and sawdust are biodegradable which may be a desirable aspect.

Woodchips, as used in the example, appear to possess considerable merit as an acceptable form of insulation. They would be manufactured near or on site with a commercial wood chipper using trees cleared from the right of way. However, careful assessment must be made of the quantity of trees available for this purpose. For example, a 100 foot wide right of way might require 9 in. of wood chips for insulation. One hundred lineal feet of this right of way would require 7,500 ft^3. At 17 pcf, 12,750 lbs. of wood chips are required. Assuming green spruce or poplar weighs approximately 40 pcf, then 3,200 ft^3 of wood are required. If a cord of wood has a porosity of 20%, then the volume of solids per cord of wood is approximately 100 ft^3. Therefore, 32 cords of wood per 10,000 sq. feet of area are required. This is a substantial amount of wood and forestry personnel should be consulted regarding estimates of the availability of these quantities in the areas where they are required.

Environmental restrictions will generally make the on-site mining of peat moss impractical. If the material is gathered locally, it must be dried to obtain a suitable moisture content and correspondingly low coefficient of thermal conductivity. Transporting processed peat moss from southern Canada to northern regions is generally considered to be impractical when compared with artificial insulation materials. It, therefore, seems reasonable to conclude that wood chips, planer shavings or styrofoam board are the three prime contenders in the choice of insulation.

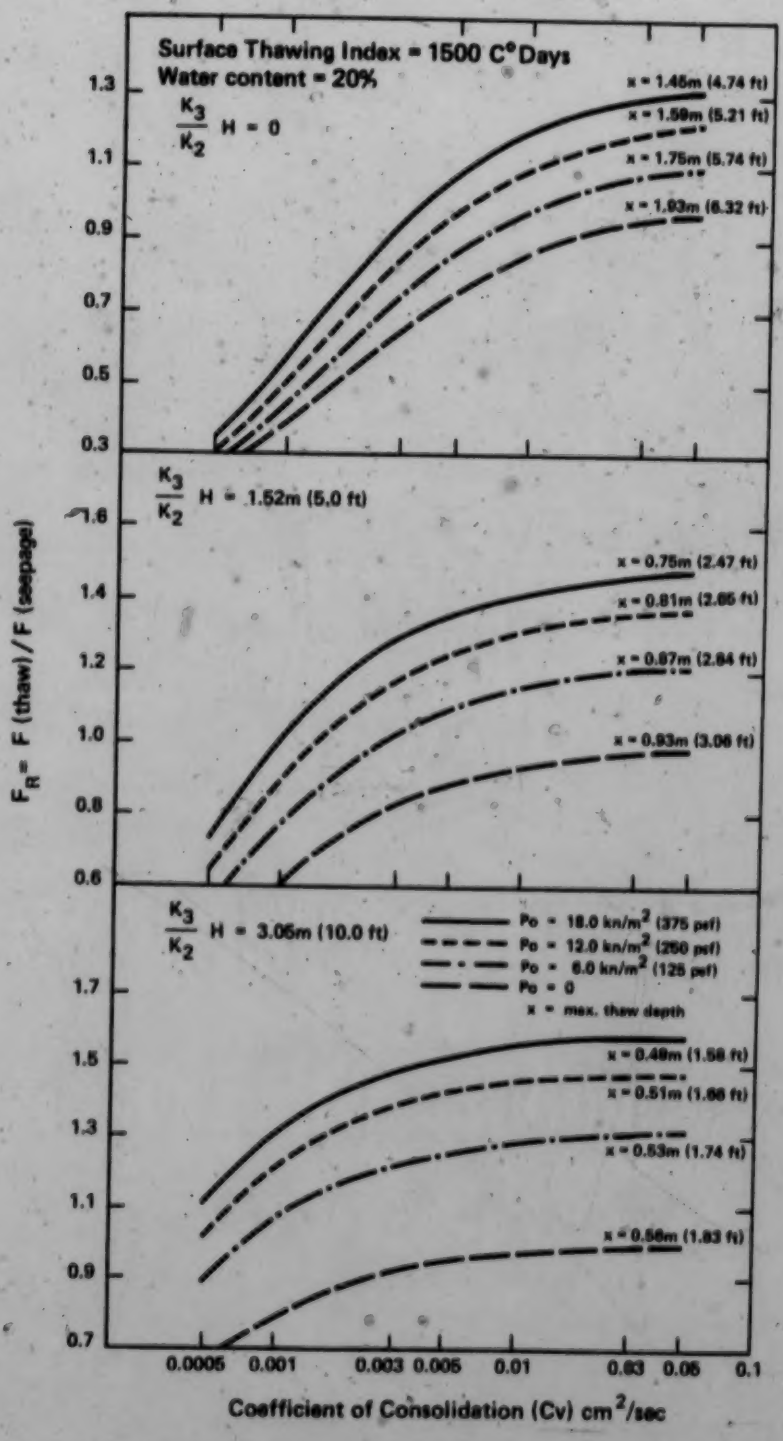
Surface Thawing Index = 1500 C° Days
Water content = 20%
K3/K2 H = 0
x = 1.46m (4.74 ft)
x = 1.59m (5.21 ft)
x = 1.75m (5.74 ft)
x = 1.93m (6.32 ft)
K3/K2 H = 1.52m (5.0 ft)
x = 0.75m (2.47 ft)
x = 0.81m (2.65 ft)
x = 0.87m (2.84 ft)
x = 0.93m (3.06 ft)
K3/K2 H = 3.05m (10.0 ft)
Po = 18.0 kn/m2 (375 psf)
Po = 12.0 kn/m2 (250 psf)
Po = 6.0 kn/m2 (125 psf)
Po = 0
x = max. thaw depth
x = 0.48m (1.58 ft)
x = 0.51m (1.66 ft)
x = 0.53m (1.74 ft)
x = 0.56m (1.83 ft)
FR = F (thaw) / F (seepage)
Coefficient of Consolidation (Cv) cm2/sec
0.0005 0.001 0.003 0.005 0.01 0.03 0.05 0.1

Fig. 3.6 Design Chart for Stabilization of Planar Landslides

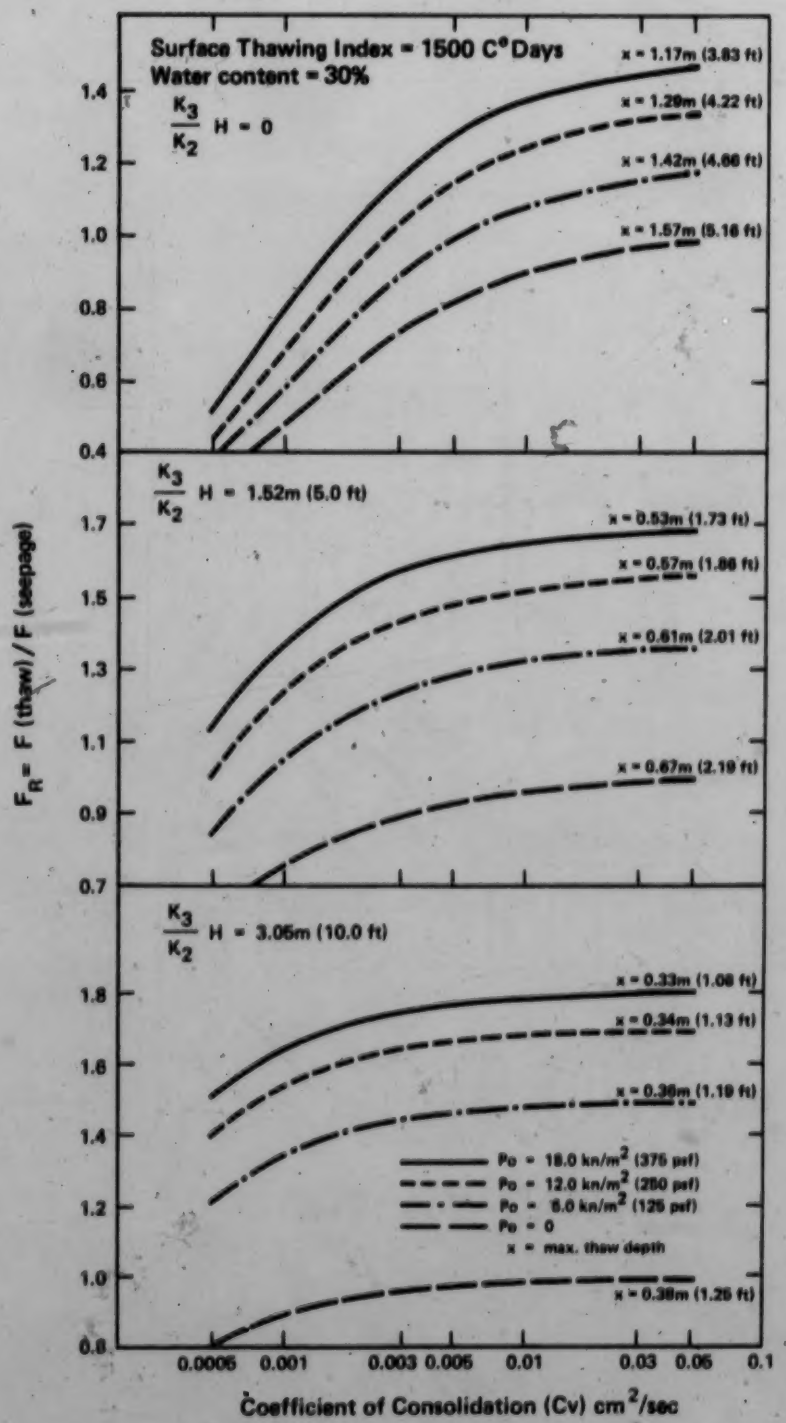
Surface Thawing Index = 1500 C°Days
Water content = 30%
K3/K2 H = 0
x = 1.17m (3.83 ft)
x = 1.29m (4.22 ft)
x = 1.42m (4.66 ft)
x = 1.57m (5.16 ft)
K3/K2 H = 1.52m (5.0 ft)
x = 0.53m (1.73 ft)
x = 0.57m (1.86 ft)
x = 0.61m (2.01 ft)
x = 0.67m (2.19 ft)
K3/K2 H = 3.05m (10.0 ft)
x = 0.33m (1.08 ft)
x = 0.34m (1.13 ft)
x = 0.36m (1.19 ft)
x = 0.38m (1.26 ft)
Po = 18.0 kn/m2 (375 psf)
Po = 12.0 kn/m2 (250 psf)
Po = 6.0 kn/m2 (125 psf)
Po = 0
x = max. thaw depth
FR = F (thaw) / F (seepage)
Coefficient of Consolidation (Cv) cm2/sec
0.0005 0.001 0.003 0.005 0.01 0.03 0.05 0.1

Fig. 3.7 Design Charts for Stabilization of Planar Landslides

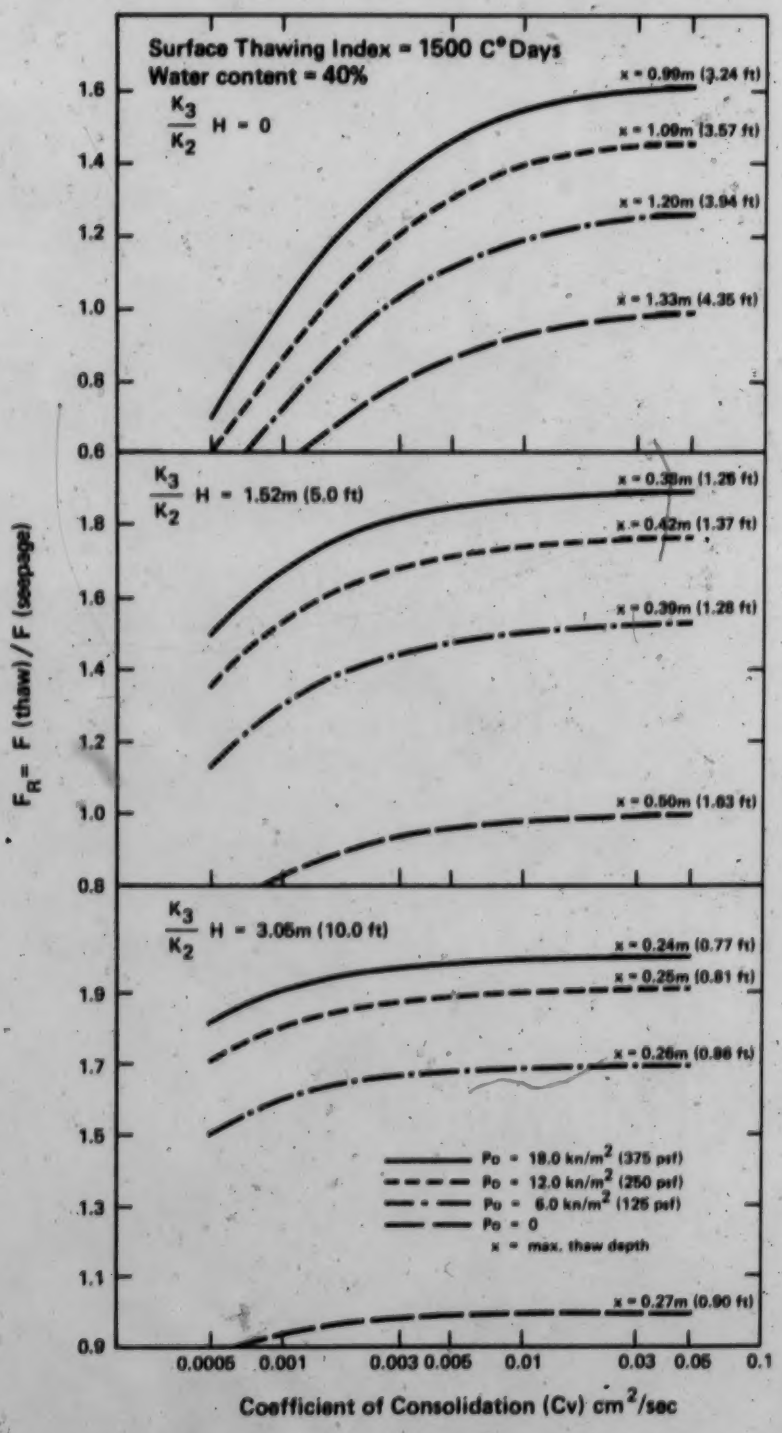
Surface Thawing Index = 1500 C° Days
Water content = 40%
K3/K2 H = 0
x = 0.99m (3.24 ft)
x = 1.09m (3.57 ft)
x = 1.20m (3.94 ft)
x = 1.33m (4.35 ft)
K3/K2 H = 1.52m (5.0 ft)
x = 0.38m (1.25 ft)
x = 0.42m (1.37 ft)
x = 0.39m (1.28 ft)
x = 0.50m (1.63 ft)
K3/K2 H = 3.05m (10.0 ft)
x = 0.24m (0.77 ft)
x = 0.25m (0.81 ft)
x = 0.26m (0.86 ft)
x = 0.27m (0.90 ft)
Po = 18.0 kn/m2 (375 psf)
Po = 12.0 kn/m2 (250 psf)
Po = 6.0 kn/m2 (125 psf)
Po = 0
x = max. thaw depth
FR = F (thaw) / F (seepage)
Coefficient of Consolidation (Cv) cm2/sec
0.0005
0.001
0.003
0.005
0.01
0.03
0.05
0.1

Fig. 3.8 Design Charts for Stabilization of Planar Landslides

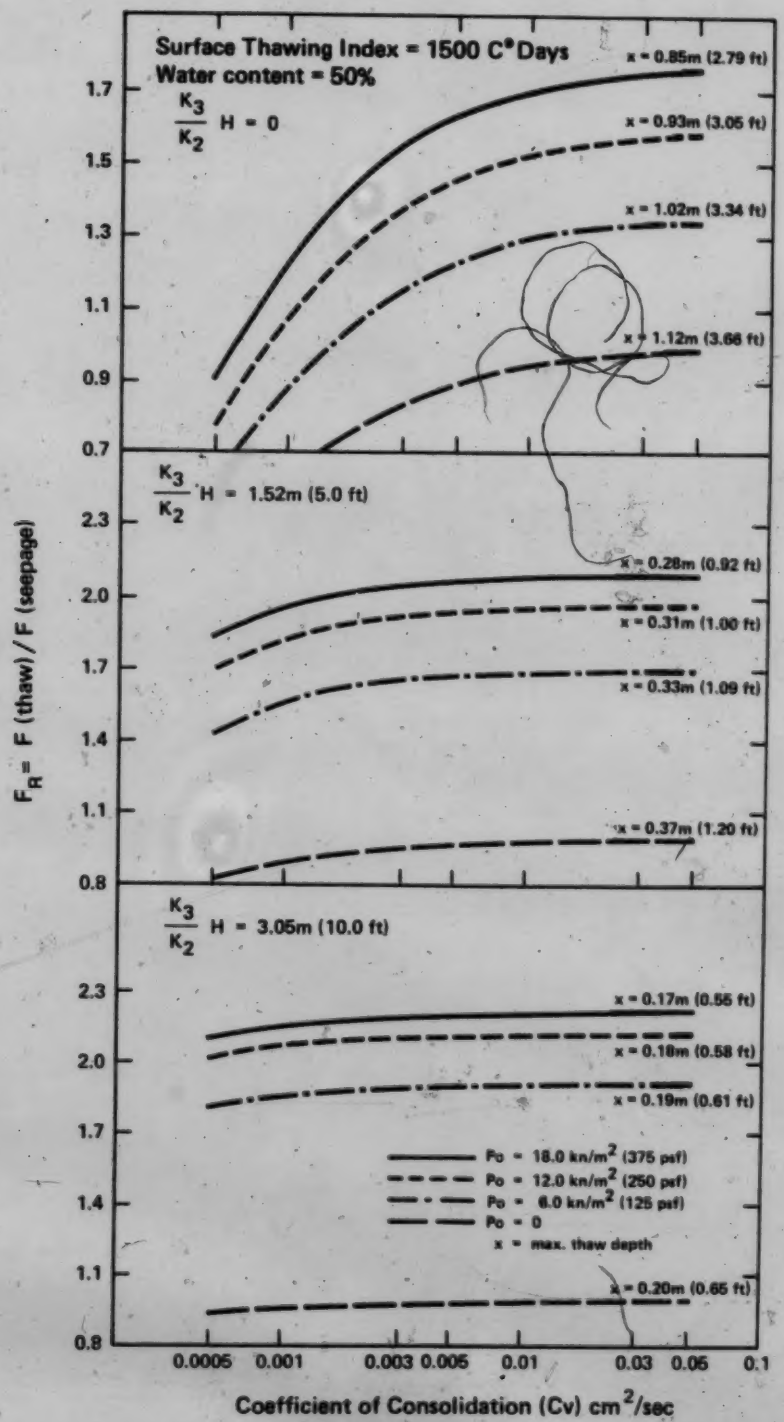
Surface Thawing Index = 1500 C°Days
Water content = 50%
K3/K2 H = 0
x = 0.85m (2.79 ft)
x = 0.93m (3.05 ft)
x = 1.02m (3.34 ft)
x = 1.12m (3.68 ft)
K3/K2 H = 1.52m (5.0 ft)
x = 0.28m (0.92 ft)
x = 0.31m (1.00 ft)
x = 0.33m (1.09 ft)
x = 0.37m (1.20 ft)
K3/K2 H = 3.05m (10.0 ft)
x = 0.17m (0.55 ft)
x = 0.18m (0.58 ft)
x = 0.19m (0.61 ft)
x = 0.20m (0.65 ft)
Po = 18.0 kn/m2 (375 psf)
Po = 12.0 kn/m2 (250 psf)
Po = 6.0 kn/m2 (125 psf)
Po = 0
x = max. thaw depth
FR = F (thaw) / F (seepage)
Coefficient of Consolidation (Cv) cm2/sec
0.0005 0.001 0.003 0.005 0.01 0.03 0.05 0.1

Fig. 3.9 Design Charts for Stabilization of Planar Landslides

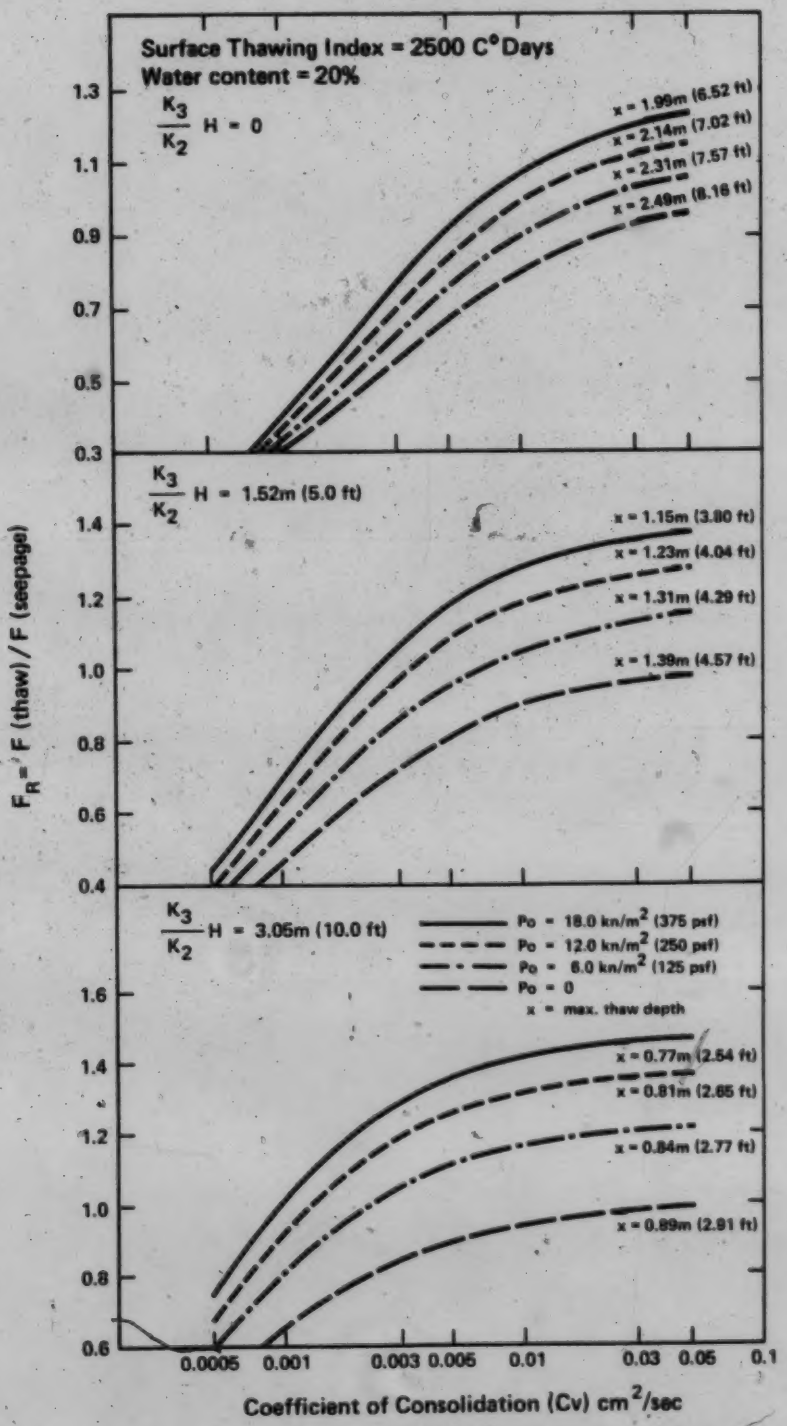

Fig. 3.10 Design Charts for Stabilization of Planar Landslides

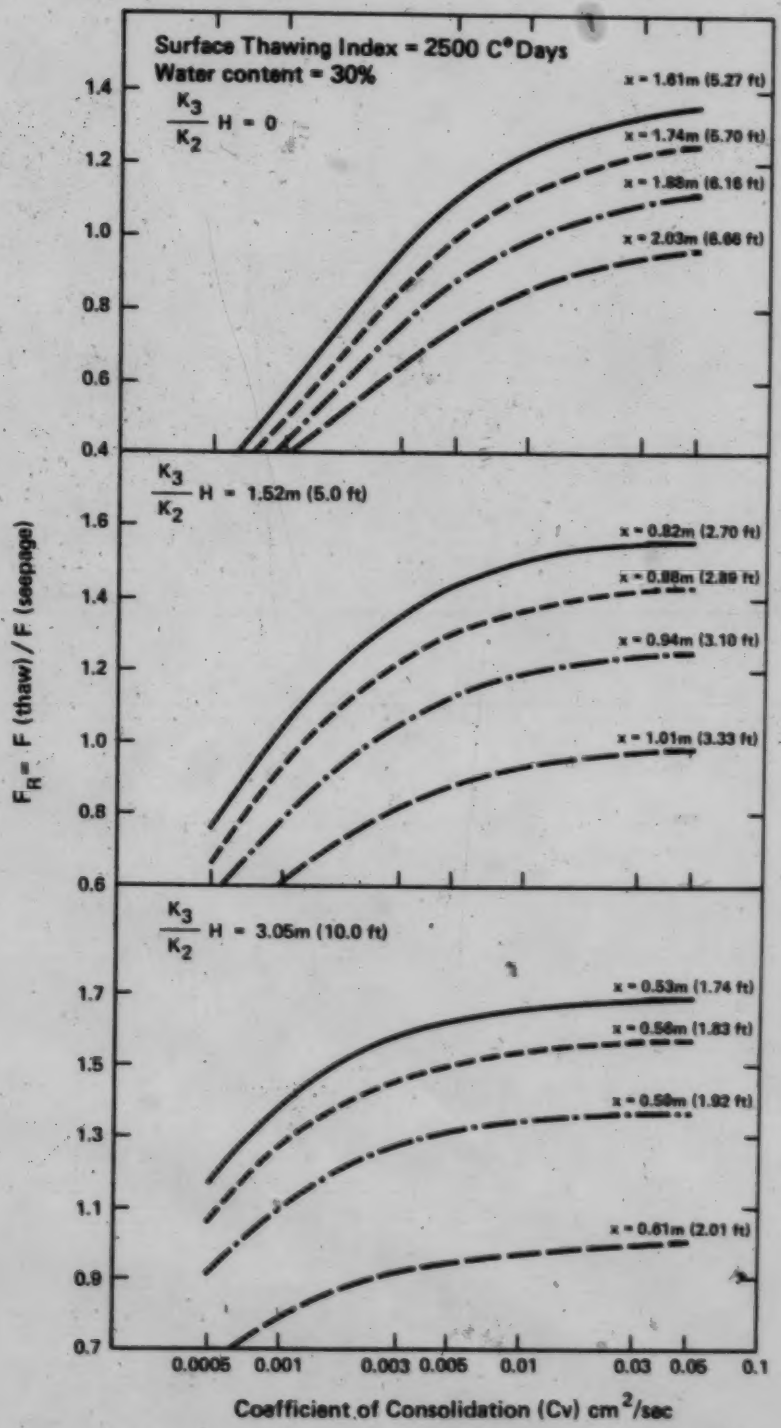
Surface Thawing Index = 2500 C° Days
Water content = 30%
K3/K2 H = 0
x = 1.61m (5.27 ft)
x = 1.74m (5.70 ft)
x = 1.88m (6.16 ft)
x = 2.03m (6.66 ft)
K3/K2 H = 1.52m (5.0 ft)
x = 0.82m (2.70 ft)
x = 0.88m (2.89 ft)
x = 0.94m (3.10 ft)
x = 1.01m (3.33 ft)
K3/K2 H = 3.05m (10.0 ft)
x = 0.53m (1.74 ft)
x = 0.58m (1.83 ft)
x = 0.58m (1.92 ft)
x = 0.61m (2.01 ft)
FR = F (thaw) / F (seepage)
Coefficient of Consolidation (Cv) cm²/sec
0.0005
0.001
0.003
0.005
0.01
0.03
0.05
0.1

Fig. 3.11 Design Charts for Stabilization of Planar Landslides

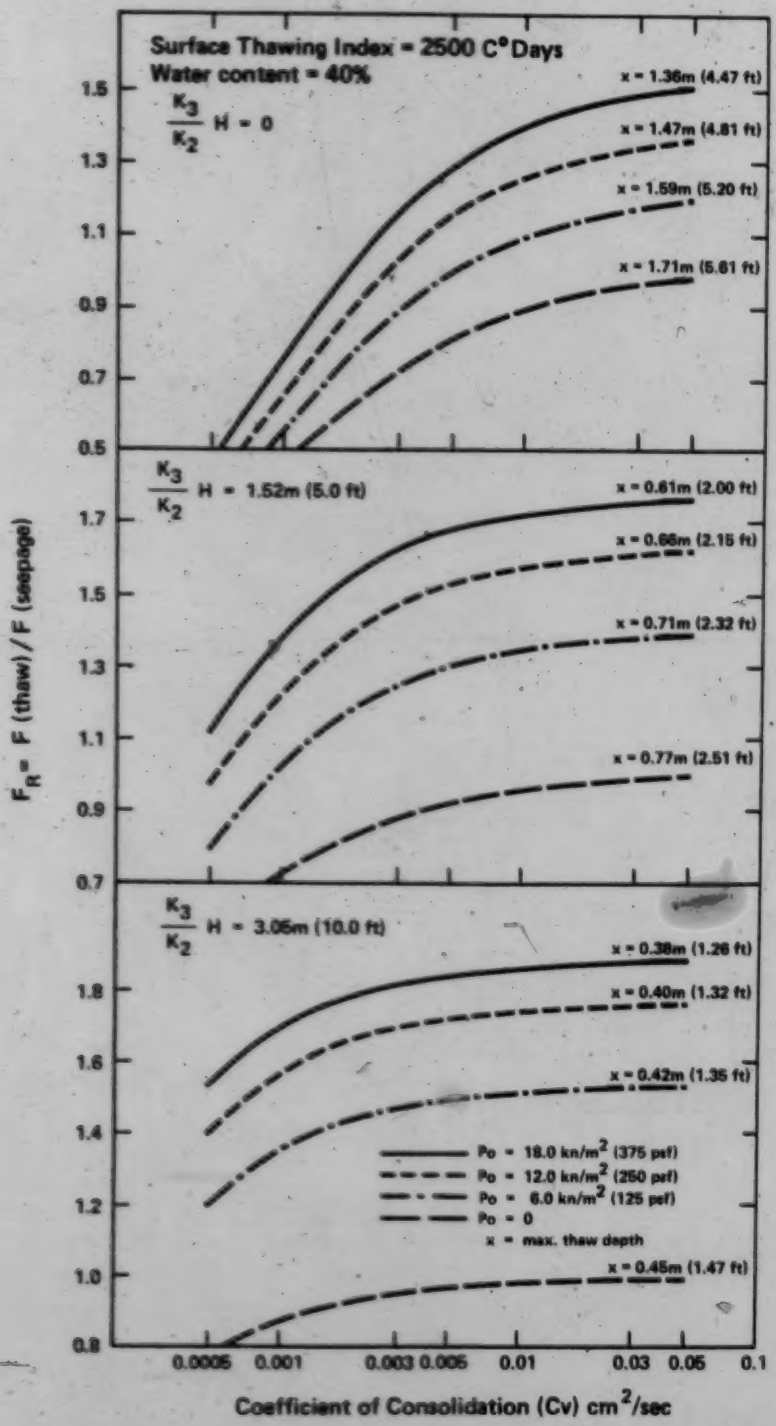
Surface Thawing Index = 2500 C° Days
Water content = 40%
K3/K2 H = 0
x = 1.36m (4.47 ft)
x = 1.47m (4.81 ft)
x = 1.59m (5.20 ft)
x = 1.71m (5.61 ft)
K3/K2 H = 1.52m (5.0 ft)
x = 0.61m (2.00 ft)
x = 0.66m (2.15 ft)
x = 0.71m (2.32 ft)
x = 0.77m (2.51 ft)
K3/K2 H = 3.05m (10.0 ft)
x = 0.38m (1.26 ft)
x = 0.40m (1.32 ft)
x = 0.42m (1.35 ft)
Po = 18.0 kn/m2 (375 psf)
Po = 12.0 kn/m2 (250 psf)
Po = 6.0 kn/m2 (125 psf)
Po = 0
x = max. thaw depth
x = 0.45m (1.47 ft)
FR = F (thaw) / F (seepage)
Coefficient of Consolidation (Cv) cm2/sec
0.0005 0.001 0.003 0.005 0.01 0.03 0.05 0.1

Fig. 3.12 Design Charts for Stabilization of Planar Landslides

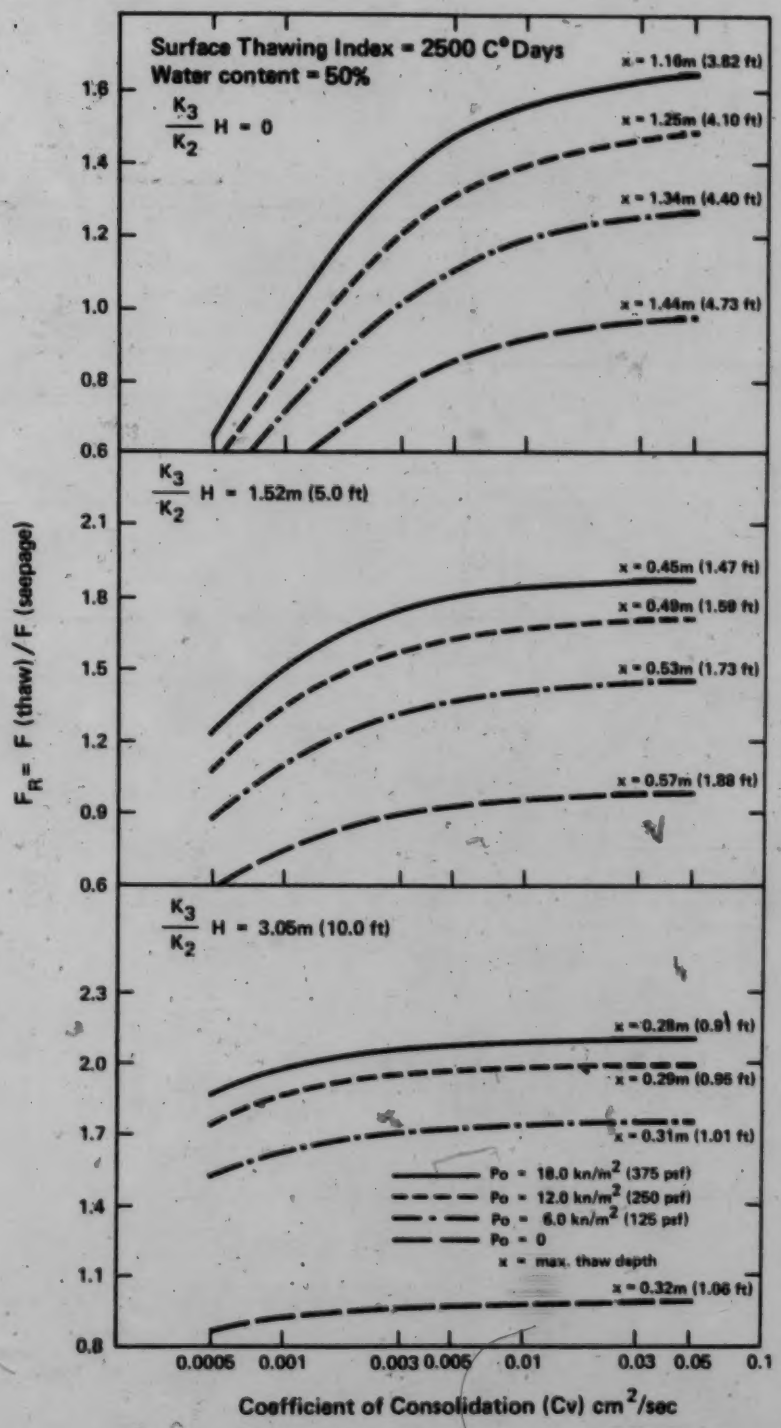
Surface Thawing Index = 2500 C° Days
Water content = 50%
K3/K2 H = 0
x = 1.16m (3.82 ft)
x = 1.25m (4.10 ft)
x = 1.34m (4.40 ft)
x = 1.44m (4.73 ft)
K3/K2 H = 1.52m (5.0 ft)
x = 0.45m (1.47 ft)
x = 0.49m (1.58 ft)
x = 0.53m (1.73 ft)
x = 0.57m (1.88 ft)
K3/K2 H = 3.05m (10.0 ft)
x = 0.28m (0.91 ft)
x = 0.29m (0.95 ft)
x = 0.31m (1.01 ft)
Po = 18.0 kn/m2 (375 psf)
Po = 12.0 kn/m2 (250 psf)
Po = 6.0 kn/m2 (125 psf)
Po = 0
x = max. thaw depth
x = 0.32m (1.06 ft)
FR = F (thaw) / F (seepage)
Coefficient of Consolidation (Cv) cm2/sec

Fig. 3.13 Design Charts for Stabilization of Planar Landslides

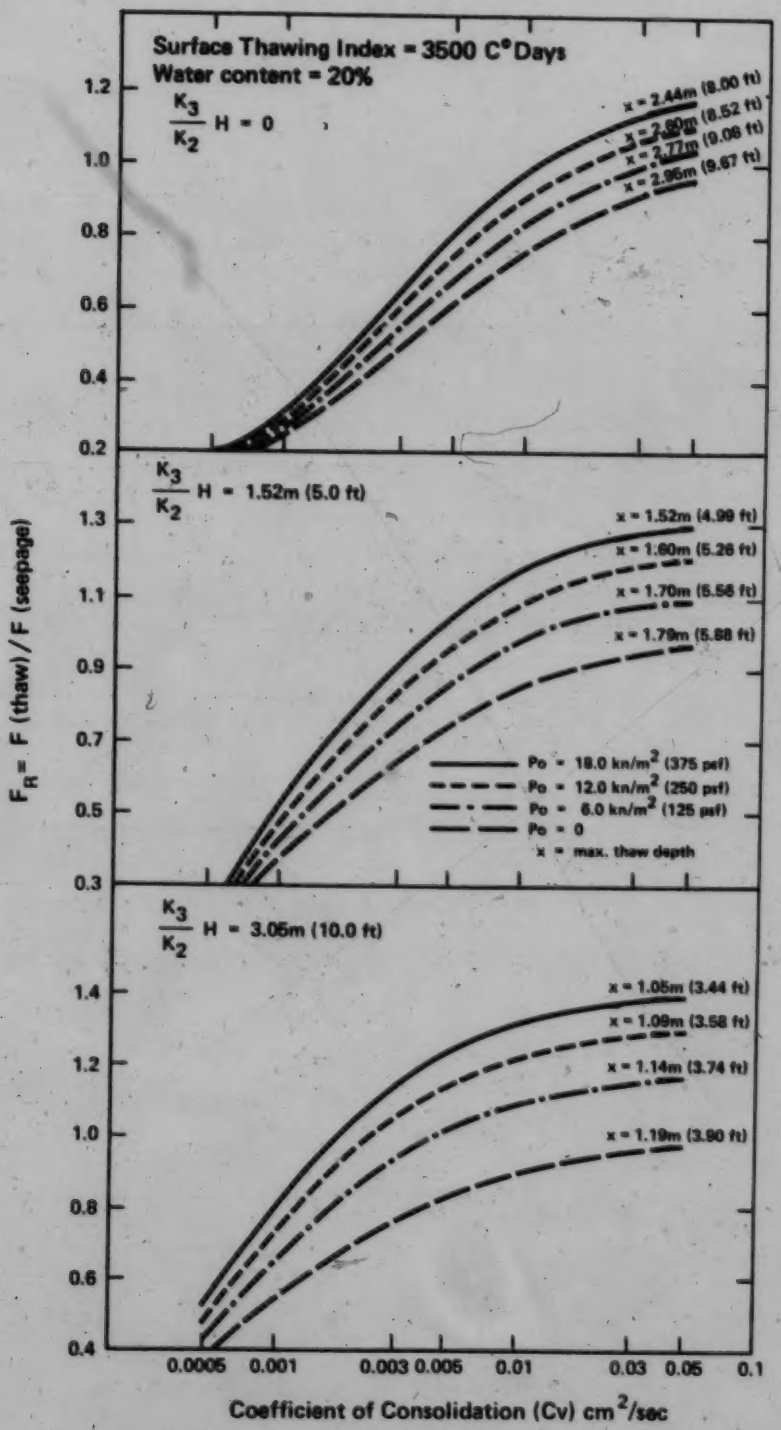
Surface Thawing Index = 3500 C° Days
Water content = 20%
K3/K2 H = 0
x = 2.44m (8.00 ft)
x = 2.60m (8.52 ft)
x = 2.77m (9.08 ft)
x = 2.96m (9.67 ft)
K3/K2 H = 1.52m (5.0 ft)
x = 1.52m (4.99 ft)
x = 1.60m (5.26 ft)
x = 1.70m (5.56 ft)
x = 1.79m (5.88 ft)
Po = 18.0 kn/m2 (375 psf)
Po = 12.0 kn/m2 (250 psf)
Po = 6.0 kn/m2 (125 psf)
Po = 0
x = max. thaw depth
K3/K2 H = 3.05m (10.0 ft)
x = 1.05m (3.44 ft)
x = 1.09m (3.58 ft)
x = 1.14m (3.74 ft)
x = 1.19m (3.90 ft)
FR = F (thaw) / F (seepage)
Coefficient of Consolidation (Cv) cm2/sec
0.0005
0.001
0.003
0.005
0.01
0.03
0.05
0.1

Fig. 3.14 Design Charts for Stabilization of Planar Landslides

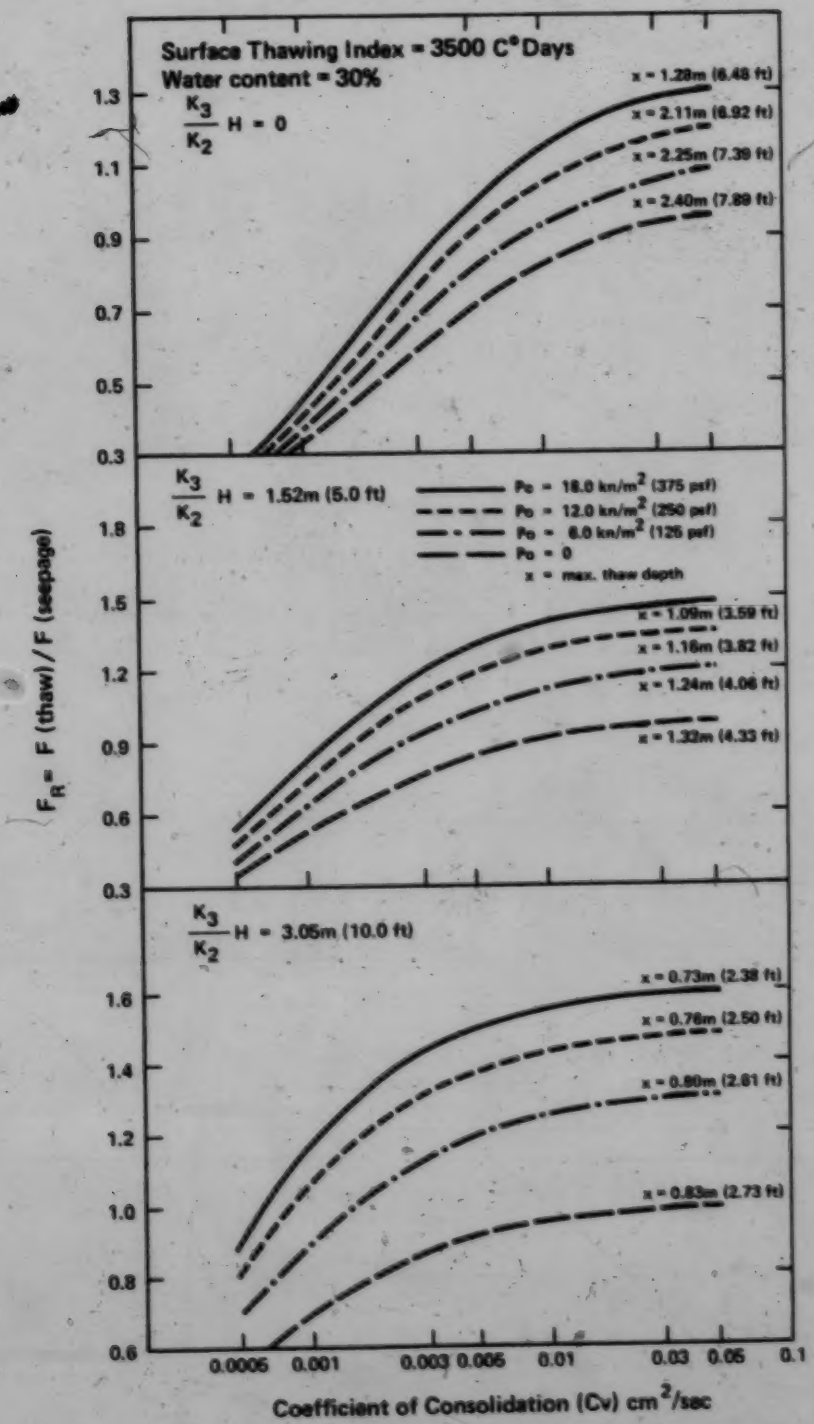
Surface Thawing Index = 3500 C° Days
Water content = 30%
K3/K2 H = 0
x = 1.28m (6.48 ft)
x = 2.11m (6.92 ft)
x = 2.25m (7.39 ft)
x = 2.40m (7.89 ft)
K3/K2 H = 1.52m (5.0 ft)
Po = 18.0 kn/m2 (375 psf)
Po = 12.0 kn/m2 (250 psf)
Po = 6.0 kn/m2 (125 psf)
Po = 0
x = max. thaw depth
x = 1.09m (3.59 ft)
x = 1.16m (3.82 ft)
x = 1.24m (4.06 ft)
x = 1.32m (4.33 ft)
K3/K2 H = 3.05m (10.0 ft)
x = 0.73m (2.38 ft)
x = 0.78m (2.50 ft)
x = 0.80m (2.61 ft)
x = 0.83m (2.73 ft)
FR = F (thaw) / F (seepage)
0.0005 0.001 0.003 0.005 0.01 0.03 0.05 0.1
Coefficient of Consolidation (Cv) cm2/sec

Fig. 3.15 Design Charts for Stabilization of Planar Landslides

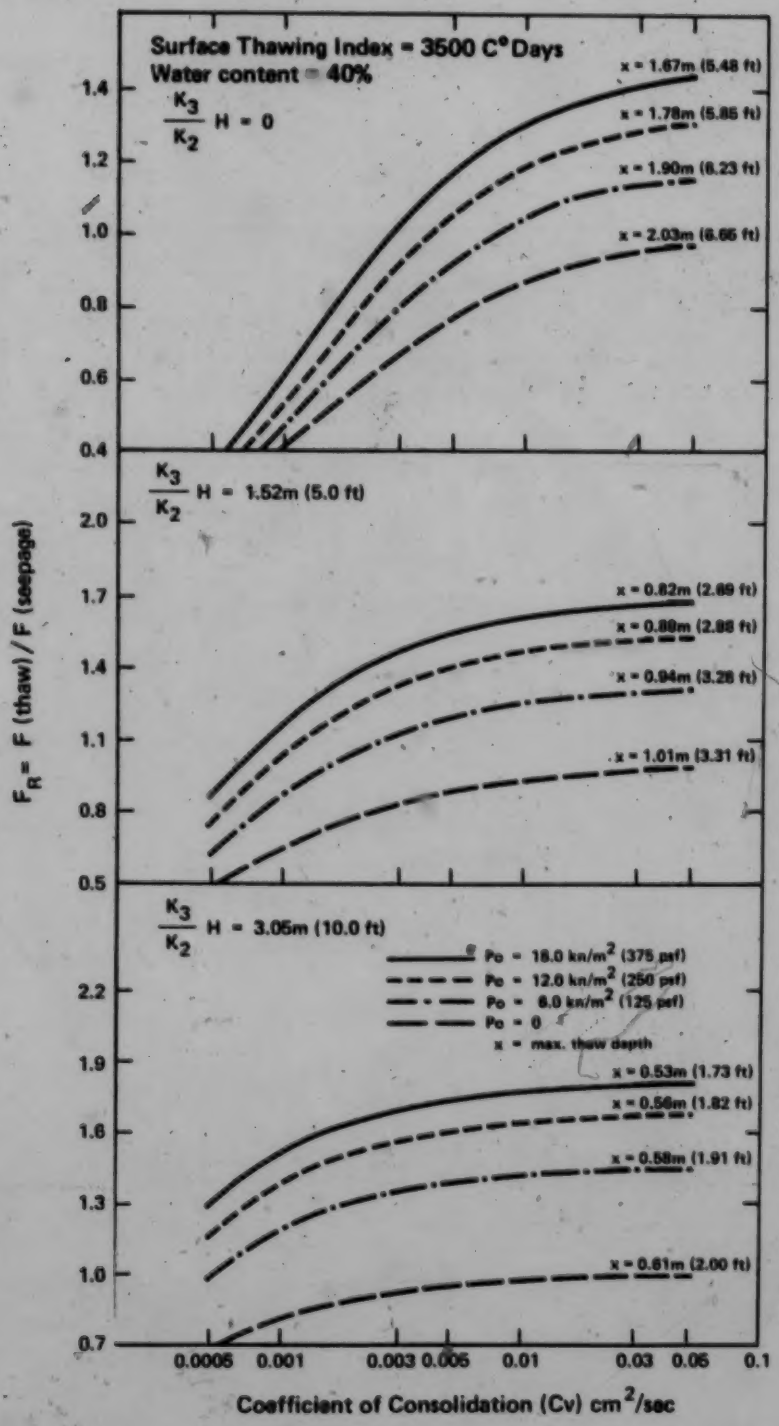
Surface Thawing Index = 3500 C° Days
Water content = 40%
K3/K2 H = 0
x = 1.67m (5.48 ft)
x = 1.78m (5.85 ft)
x = 1.90m (6.23 ft)
x = 2.03m (6.66 ft)
K3/K2 H = 1.52m (5.0 ft)
x = 0.82m (2.69 ft)
x = 0.89m (2.88 ft)
x = 0.94m (3.28 ft)
x = 1.01m (3.31 ft)
K3/K2 H = 3.05m (10.0 ft)
Po = 18.0 kn/m2 (375 psf)
Po = 12.0 kn/m2 (250 psf)
Po = 6.0 kn/m2 (125 psf)
Po = 0
x = max. thaw depth
x = 0.53m (1.73 ft)
x = 0.56m (1.82 ft)
x = 0.58m (1.91 ft)
x = 0.61m (2.00 ft)
FR = F (thaw) / F (seepage)
Coefficient of Consolidation (Cv) cm2/sec
0.0005
0.001
0.003
0.005
0.01
0.03
0.05
0.1

Fig. 3.16 Design Charts for Stabilization of Planar Landslides

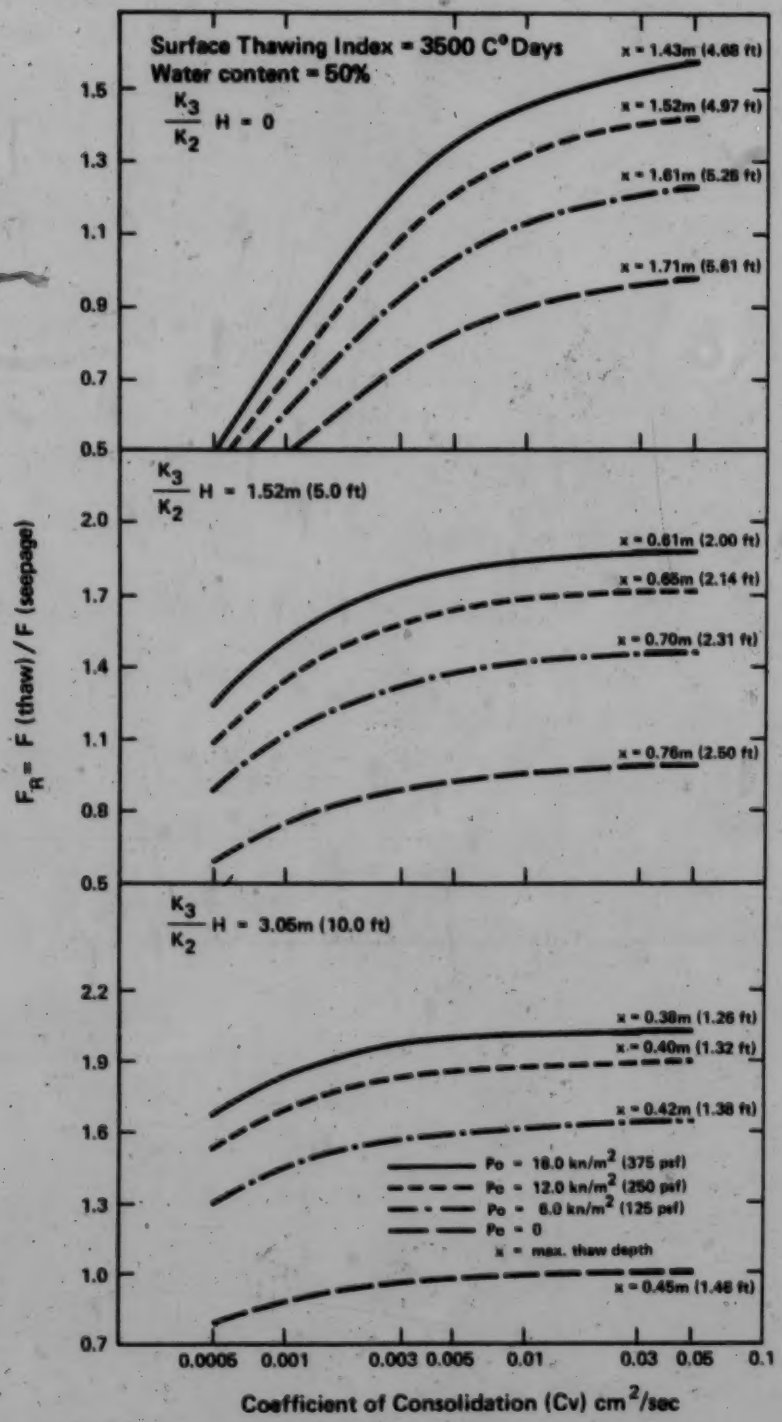

Fig. 3.17 Design Charts for Stabilization of Planar Landslides

CHAPTER IV

THE ENERGY BALANCE

4.1 Basic Concepts

The energy balance at the surface of the earth may be expressed in terms of a mathematical equation:

$$q_{DS} + q_{dS} + q_{RS} + q_{LU} + q_{LD} + q_L + q_H + q_P + q_G = 0 \quad - - - [4.1]$$

The symbol q represents energy per unit of area per unit of time and has units of gm. cal/cm^2 min. or simply ly/min. The subscript refers to the source of this energy. The equation is written with all terms positive. If the direction of energy flow is toward the surface of the ground, the sign will be positive and if energy flow is directed away from the surface, the sign will be negative. This convention also applies to heat flow below the surface. In equation [4.1]:

q_{DS} = direct beam short wave energy from the sun.

q_{dS} = diffuse short wave energy from the sky.

q_{RS} = short wave radiation reflected by the earth's surface.

q_{LD} = long wave radiation directed downward from the atmosphere to the ground.

q_{LU} = long wave radiation emitted by the ground surface.

q_L = the transfer of energy to or from the surface through the latent heat of vaporization or condensation.

q_H = the transfer of energy by sensible heat to or from the surface by turbulent or convective processes.

q_P = the gain or loss of energy through the addition of sensible heat due to precipitation on the surface.

q_G = the transfer of energy to or from the surface through the ground.

4.2 Radiative Components on a Horizontal Surface

4.2.1 General

All bodies radiate thermal energy, the intensity and spectral distribution of which is a function of the temperature and emissivity of the body.

A "black body" is a body or object that completely absorbs all radiation (at all angles) incident upon its surface. It is also a perfect emitter at all wave lengths. The emissivity (ϵ) of a body is the ratio of the emissive power (E) of a body at absolute temperature (T) to the emissive power of a black body (E_b) at the same absolute temperature (T). Thus,

$$\epsilon = E/E_b \quad - - - [4.2]$$

Radiation is characterized according to wave length (λ) usually measured in microns (μ). The spectral distribution ranges from some small, but finite value (.10μ) to wave lengths in excess of 50μ.

Planck (1959) used quantum theory to derive the hemispherical emissive power of a black body.

$$E_{b\lambda} = \frac{C_1}{\lambda^5 (e^{C_2/\lambda T} - 1)} \quad - - - [4.3]$$

where C_1 and C_2 are constants and $E_{b\lambda}$ is the emissive power at wave length λ and absolute temperature (T) expressed in ^oK. Figure 4.1 indicates the manner in which black bodies emit energy at different temperatures and wave lengths. It also shows that the maximum energy emitted by a black body occurs at some unique wave length

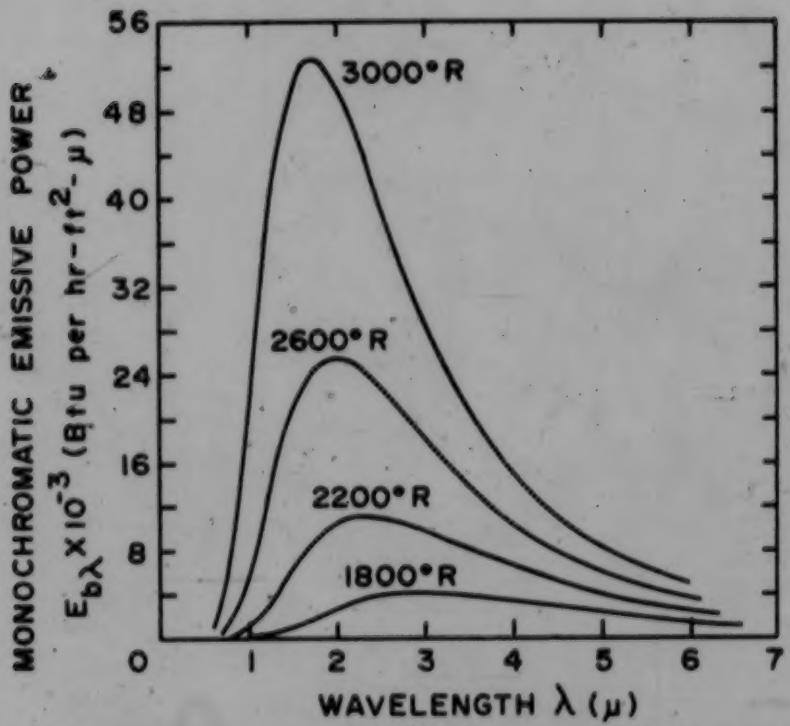

Fig. 4.1 Monochromatic Emissive Power for Several Temperatures vs Wavelength (After Wiebelt (1966))

associated with some unique temperature. If equation [4.3] is differentiated partially with respect to wave length and set equal to zero, the equation $\lambda_{max} T$ = constant emerges where λ_{max} is the wave length corresponding to the maximum emissive power (Wien, 1894). The Wien's Displacement Law indicates that the peak intensity of emitted radiation occurs at a wave length which is inversely proportional to the absolute temperature. It is evident that so called "hot bodies" radiate maximum amounts of energy at short wave lengths and cooler bodies radiate at longer wave lengths. The sun, at approximately 6,000^0 K, has its peak emissive power in the 0.47 to 0.49 μ range, whereas the earth at approximately 300^0 K emits at a peak intensity of 11 to 12 μ. It is of interest to note that 45% of the emitted energy in the solar spectrum falls within the visible range of 0.4 to 0.74 μ (Sellers, 1965). Short wave radiation is generally

considered to be all that radiation less than 4 μ in length.

The Planck equation for emissive power in monochromatic wave lengths may be recast and integrated over all wave lengths to form the Stefan-Boltzman equation.

$$E = \sigma T^4 \quad - - - [4.4]$$

This equation describes the energy emitted by a black body in all wave lengths at absolute temperature (T) and where the Stefan-Boltzman constant (σ) equals 8.26×10^{-11} ly/min. $^{o}K^{4}$. If the emissivity (ε) is included, then the equation describes the manner in which bodies other than black bodies emit radiation.

4.2.2 Direct Beam Clear Sky Solar Radiation

As solar radiation passes through the atmosphere it is selectively scattered and/or absorbed by gases and impurities in the air. Ozone absorbs all the energy at wave lengths less than 0.3 μ and selectively up to 0.35 μ. Water vapour effectively prevents solar radiation longer than 4.0 μ from reaching the ground. It selectively transmits in the 11 to 13 μ range and this accounts for the significant influence of long wave radiation.

Carbon dioxide is the only other gas in the atmosphere that absorbs radiation. It will, however, absorb only in two narrow bands. The first band occurs between 0.29 and 4.0 μ and the second between 12 and 16 μ (Simpson, 1929).

Actual depletion of a direct ray of radiation is a function of the composition and thickness of the medium through which it passes. The quantity of direct beam radiation (on a plane normal to the beam) reaching the earth's surface is commonly expressed by Beer's Law.

$$q_{DSN} = J_o e^{-\beta m} \quad - - - [4.5]$$

where q_{DSN} is the flux of energy transmitted to the earth by direct short wave radiation.

β = coefficient of extinction.

m = thickness of the medium traversed by the beam or commonly called the optical air mass.

This quantity m may be adequately approximated by $^1/\cos z$ (where z equals the zenith angle of the sun) for angles of z less than 80 degrees. The solar constant (J_o) is defined as the intensity of solar radiation received on a unit area of a plane normal to the incident radiation at the outer limit of the earth's atmosphere with the earth at its mean distance from the sun. The magnitude of this constant varies within limits of ± 1.5%. List (1968) proposes a value of 1.94 ly/min. Occasionally adjustments are made to J_o that take the form of J_o/r^2 where r, the radius vector of the earth, is the distance from the center of the earth to the center of the sun expressed in terms of the length of the semi-major axis of the earth's orbit (ibid). This variation in distance produces a variation in intensity of ± 3.5%.

The term $e^{-\beta}$ is alternately called the transmittance (a). This parameter is a function of scattering due to air molecules and water vapour as well as absorption of radiation by water vapour. The amount of radiation reaching a horizontal plane on the earth's surface may then be written as

$$q_{DSH} = J_o a^m \cos z \quad - - - [4.6]$$

where z again represents the zenith angle of the sun and can readily be expressed in terms of the declination, latitude and the hour angle.